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AMERICAN SOCIETY OF CIVIL ENGINEERS
INSTITUTED 1852

PAPERS AND DISCUSSIONS

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ENGINEERING AND ENGINEERING EDUCATION

ADDRESS AT THE ANNUAL CONVENTION, AT MILWAUKEE, WIS., JULY 10, 1929

BY ANSON MARSTON,* PRESIDENT, AM. SOC. C. E.

This address will be devoted to the presentation:

First, of my definition of engineering, accompanied by a brief discussion of its component phrases and of the nature of the Engineering Profession; and

Second, of my opinion and forecast that the trend of engineering education during the next quarter century should and probably will develop active co-operation and, to some extent, responsible direction by our National Engineering Societies.

A DEFINITION OF ENGINEERING

In 1827, the Council of the Institution of Civil Engineers requested Thomas Tredgold to "give a description of what a civil engineer is", to be incorporated in a petition for a charter. The resulting definition was:

"Civil Engineering is the art of directing the great sources of power in Nature for the use and convenience of man, as the means of production and of traffic in states both for external and internal trade, as applied in the construction of roads, bridges, aqueducts, canals, river navigation and docks, for internal intercourse and exchange, * * *, and in the construction and adaptation of machinery, and in the drainage of cities and towns."

The many attempts made by engineers during the last forty years to formulate a new definition of engineering prove that this famous statement incorporated in 1827 in the charter of the (British) Institution of Civil Engineers is not satisfactory under present-day conditions.

The difficulties in the way of devising a satisfactory definition of modern engineering are very great, perhaps insuperable. Many objections, at least some of which appear valid, have been raised to every suggested definition of which I know.

* Dean of Eng., Iowa State Coll., Ames, Iowa.

In 1927, the Mining and Metallurgical Society of America, presented an original and an amended definition of engineering,* both prepared by a committee, together with the opinions thereon of 95 members. The definitions were as follows:

Original Definition:

"Engineering is the practical application of scientific methods to the utilization of the resources of Nature for the use of Man."

Amended Definition:

"Engineering is the application of scientific methods to the utilization of the resources of Nature."

Only 7 members approved the original definition and only 34 the amended definition, while 44 members were not satisfied with either. Many thought the definitions basically wrong, because they would include many occupations other than engineering.

In the years 1906-1919, inclusive, I gave annually to engineering seniors a course of lectures, about half of which were printed and copyrighted in 1912, on the History of Engineering, containing some discussions on definitions of engineering. In 1913-1914, I served as Chairman of a committee which made a study of original source material and formulated definitions of mechanic arts and of engineering, which were adopted officially and published in 1914 by the Land Grant College Engineering Association.

I was never entirely satisfied with this 1914 definition of engineering, and have continued intermittent study of the subject until now I wish to suggest another form, as follows:

"Engineering is the science and art of directing applications of the science of mechanics in the economic utilization of the forces and materials of Nature."

The significant phrases in this suggested definition of engineering are: "Science and Art"; "Science and Art of Directing"; "Applications of the Science of Mechanics"; "Economic Utilization"; "Utilization of the Forces and Materials of Nature"; and "Forces and Materials of Nature". The definition is most readily explained by discussing these phrases in reverse order.

The Forces and Materials of Nature.—This phrase covers the entire physical universe and, hence, it includes the entire field of the physical sciences and applied mathematics. While engineering is founded mainly on the sciences of mathematics, physics, and chemistry, in many instances it is based on biological science.

Engineering, however, does not include all applications of the physical and mathematical sciences, but is restricted to the particular ones that are characterized by the necessary use of designs, methods, and devices which are based on the science of mechanics.

It is a common fault of suggested definitions of engineering to word them so broadly that while often they may describe engineering with much truth, they include various other occupations which are widely different.

* *Bulletin 190*, Min. and Metallurgical Soc. of America, October, 1927.

The Utilization of the Forces and Materials of Nature.—This phrase covers not only engineering, but agriculture, the handicrafts, medicine; in short, all industries, all trades, and all professions, in which any use is made of any of the organic or inorganic forces or materials of the physical universe.

Engineering, however, does not include all utilizations of the forces and materials (the resources) of Nature, but is restricted to those particular utilizations which are characterized by applications of the science of mechanics.

Economic Utilization.—Emphasis has always been given the economic side of the engineer's work, his ability "to do with one dollar what some one else could do with two". In recent years this has been made more emphatic, and it is now widely recognized that the engineer must have a working mastery of the correct principles of economics and business. Engineering is now recognized as including the work of the industrial executive.

Applications of the Science of Mechanics.—This phrase covers all mechanic arts industries. In 1914, the Land Grant College Engineering Association officially adopted the following definition:

"The mechanic arts are those arts which are characterized by applications of the science of mechanics."

It was prepared by the Executive Committee of the Association, after a careful study of original source material, some of which, arranged in chronological order, is quoted as follows:

"The use of the term Mechanic Arts, in standard literature dates back to Francis Bacon, 1561-1626. The term was first applied to machine devices." (Letter from the Lexicographer, *Literary Digest*, 1913.)

" 'The Mechanick Art is a Science which contemplates about the quantities of moving forces, and the times in which the motion is made.' " ("Mechanick-Powers", Venturus Mandey and James Moxon, London, 1696.)

" 'Most of what are usually called the Mechanic Arts are partly mechanical and partly chemical.' " (Webster's Dictionary, 1848; copyright date, 1847; on the title-page, 1848.)

" 'Journal of the Franklin Institute of the State of Pennsylvania, for the Promotion of the Mechanic Arts, Devoted to Mechanical and Physical Science, Civil Engineering, the Arts and Manufactures, and the Recording of American and Other Patent Inventions.' " (Title of the *Journal* for 1857, Philadelphia.)

" 'Mechanics, Applicate or Applied, is a term which, strictly speaking, includes all applications of the principles of abstract mechanics to human art, * * *.' " (Encyclopedia Britannica, 1857, American Reprint, Little, Brown & Co., Boston.)

" 'The Mechanic Arts are those which comprehend the means of promoting and facilitating the necessities of existence.' " (The American Encyclopedia, 1859-63.)

" 'Mechanical Art is most easily distinguished from fine art by the character of its products: Those of fine art represent ideas, those of mechanical art answer purposes; the former result in a gratification of the æsthetical sense, the latter are made for practical use. * * *' ". (Johnson's New Universal Cyclopedia, 1877, Edited by F. A. P. Barnard, President of Columbia, and Arnold Guyot, Professor of Geology and Physical Geography, College of New Jersey.)

No permanent list of mechanic arts industries can be made, for new industries are constantly developing as civilization advances. A present-day list would include:

Ceramic Industries.
Communication Industries: Telegraph, Telephone, Wireless.
Construction Industries.
Electrical Industries.
Gas Industries.
Handicrafts: Masonry, Metal, Wood, etc.
Manufacturing Industries.
Mining Industries.
Petroleum Industries.
Power Industries: Electric, Gas, Steam, Water, Wind.
Quarry Industries.
Sanitation Industries: Sewerage, Water Supply.
Transportation Industries: Aeronautics, Highways, Navigation, Railways, Waterways.
Utilities: Bus, Electric, Gas, Heat, Railway, Water.
Various other industries not enumerated.

Agriculture is not a mechanic arts industry, although it requires the use of many mechanical devices, for agriculture is characterized by the cultivation and care of plants and animals, not by the use of tools and machines. Many other industries and occupations in which the use of mechanical contrivances is essential are excluded for similar reasons.

Only a limited fraction of the hundreds of millions of persons who are employed in the mechanic arts industries can properly be termed engineers.

The Science and Art of Directing.—This phrase restricts engineering to the professional phases of mechanic arts work.

Tredgold was right in making the phrase, "the art of directing", one of the most promising components of his famous definition, and the word, "directing", should be considered an essential part of any correct definition of engineering.

Engineering is a true profession, with real professional characteristics and requirements, including:

1.—A great system of engineering education.
2.—Increasingly widespread requirement of governmental professional licenses to practice engineering.
3.—A great professional engineering literature.
4.—Many local and several great National professional engineering societies in each great country.
5.—Increasingly strict enforcement of formal codes of professional engineering ethics.

Engineering differs in characteristics from the other great professions mainly in that the engineer is unique in his close and vital relation to hundreds of millions of persons engaged in pursuits closely allied to his profession. It is easy to make the distinction between the apothecary and the physician, or between the bailiff and the lawyer, but it is not so easy to draw a sharp line of separation between the highly skilled mechanic arts master workman and the professional engineer. This is because engineering is both a science and an art.

Science and Art.—Science is classified knowledge; "art is that which depends on practice and skill in performance". "In science, we know that we may know; in art, we know that we may produce."*

* See Webster's Dictionary, and Karslake as quoted therein.

From the remotest ages, engineering has originated from the union of the most advanced contemporaneous physical and applied mathematical science with the highest contemporaneous skill attained in practice in the mechanic arts. As far back as the beginning of the Christian era, the Roman architect-engineer, Vitruvius,* wrote of architecture, which then included engineering:

"Architecture is a science arising out of many other sciences, and adorned with much and varied learning; by the help of which a judgment is formed of those works which are the results of other arts. Practice and theory are its parents. * * * the mere practical architect is unable to assign sufficient reasons for the forms he adopts; and the theoretic architect also fails, grasping the shadow instead of the substance. He who is theoretic as well as practical, is therefore doubly armed; able not only to prove the propriety of his design but equally to carry it into execution."

In Tredgold's day (1827), almost all English engineers had risen from the ranks of master workmen, and it was natural for him to refer to engineering as only an "art". Since Tredgold's time, there has been a very extensive development of the scientific aspects of engineering knowledge and qualifications, which I have come to believe should be recognized by defining engineering as both a "science" and an "art". While engineering is based on certain phases of the physical and mathematical sciences, there is a body of real "engineering science," consisting of certain highly specialized developments (dealing almost wholly with engineering problems) of the physical and mathematical sciences. Engineering mechanics, the theory of structures, and the theory of machine design are good illustrations. The physicist, the chemist, the biologist, the mathematician, would have to become engineers to become qualified to develop and advance modern engineering science.

More and more universally in modern engineering, empirical methods and solutions of engineering problems are found insufficient and unsatisfactory. No engineering problem can be considered solved satisfactorily until a sound theory of its solution is discovered.

The origin of the words, "engineering" and "engineer", is such as to indicate that the engineer should be thought of as primarily a man of genius, or possessing ingenuity, and that engineering should not be considered as merely an art.

The words† "engineering" and "engineer" come from the Sanskrit, *jan*, to be born, from which came the Greek form, γεν, and the Latin, *gen*.

"The direct line of derivation [of the word, engineer] may be traced to *ingenium*—natural capacity, talent, capacity for invention—and is closely linked with the words *genius* and *ingenuity*."‡

The name, "engineer", came to us from the Latin through the French noun, *ingénieur*.

It is a mistake to think that the name of our profession is derived from the word, engine, or that the engineer is primarily concerned with engines. An engineer is an ingenious man, a man of genius. An engine is an ingenious machine.

* Translation by Joseph Gwilt, an English architect, 1874.

† "Life of Sir William Fairbairn," by Dr. William Pole (Abridged), *Minutes of Proceedings*, Inst. C. E., Vol. CLXXIX, 1909-10.

‡ *Bulletin No. 16*, Soc. for the Promotion of Eng. Education, 1929.

Engineering Education—Its Relation to Professional Engineering Organizations

No profession can be a true profession and keep up with the advance of science without wise, ample, and ever-improving provision for its professional educational requirements. In the United States, our engineering societies have not taken a sufficiently active part in shaping and directing engineering education. In a paper* entitled "A Comparative Study of Engineering Education in the United States and in Europe", W. E. Wickenden gives an excellent summary discussion of the development of modern engineering education in Europe and in the United States. The time has come when our engineers' clubs within the great cities, our State engineering societies within the several States, and our great National engineering societies for the Nation, ought to assume real responsibility and undertake real initiative in engineering education.

It is not sufficient for engineering practitioners to confine themselves to criticism of engineering graduates; that is too much like school boys making faces at their professors from behind their backs.

The writer agrees with Mr. Wickenden* that:

> "In no other country [than the United States] have the engineering schools been so free from outside domination. They owe little to statecraft other than the provision of means for their extension and support. They owe little to the organized engineering profession except the benefits of occasional criticism of their aims and methods. They owe little to industries except an ever-widening field of employment for their graduates. To a striking degree, these schools have been left to work out their own destiny and fend for themselves. This has been the source of both weakness and strength."

It should be borne in mind that throughout all civilizations, up to the beginnings in the Eighteenth Century, in France, of the development of our modern engineering schools, the education of engineers was left almost wholly to themselves. We know by the character of their engineering constructions that the great engineers of Ancient Chaldea and Egypt must have been fully acquainted with the best scientific and mechanics arts knowledge of their times. We know that Heron, of Alexandria, whom Truesdell† has erroneously designated "the first engineer", was a great mathematician and author, as well as a great engineer. The Roman architect-engineer Vitruvius, enumerates the subjects in which an architect should be educated, as including: Good writing (we now say "English"); history; music; moral philosophy (ethics); physics (sanitation); law (business law relating to party walls, roof water, cesspools, sewers, and the like); natural philosophy (physics, as exemplified in the works of Archimedes and Ctesibus and as applied to the conduct of water); drafting; arithmetic; geometry; and astronomy.

In modern times, engineering education in England was left mainly to the Engineering Profession itself for a long time and, according to Mr. Wickenden,*

* "A Comparative Study of Engineering Education in the United States and in Europe", *Bulletin No. 16*, Soc. for the Promotion of Eng. Education, 1929.

† *Journal*, Assoc. of Eng. Societies, Vol. 19, July, 1897.

"From its foundation in 1818, the Institution of Civil Engineers has exercised a dominant influence on the professional training of engineers. British policy is to leave such matters to the organized profession rather than the law and the professional institution, by virtue of its character, acts as a quasi-legal body."

In Germany, says Mr. Wickenden,*

"The influence of the *Verein Deutscher Ingenieure* must be ranked among the major forces in the upbuilding of both the technical universities and the *realschulen.*"

It is true that America is not England or Germany. The reason for our comparative failure to the present time to develop adequate systems of elementary trade schools and technical high schools is the universal freedom to choose any occupation and to aspire to any social plane which our American ideals strive to preserve for the American youth until they have passed the high school age. Our engineering schools are not to blame, as some have charged. Those engineering schools which have failed to devote their energies mainly to professional college courses have been killed or dwarfed by lack of attendance. You cannot teach without students.

The fact that the problem of developing adequate systems of technical education is vastly more difficult in America than in countries where class distinctions lead the great majority of youth to settle inevitably at comparatively early ages into defined life occupations, is not a valid excuse for American professional engineering societies to neglect their duty to assume proper responsibility and initiative in connection with engineering education. By co-operation with the Society for the Promotion of Engineering Education the American Society of Civil Engineers should proceed promptly to assume real responsibilities in shaping civil engineering education in our engineering colleges. In this, as in other engineering activities, the Society should work in co-operation with the other Founder Societies, as far as they are ready to go.

In my judgment, the Society might well at this time study with great care the feasibility and advisability of proceeding in the immediate future with active co-operation in shaping and advancing civil engineering education in America in accordance with all or a part of the following program, several features of which have already been given tentative consideration by the Board of Direction:

1.—Adopt formal requirements, materially higher than have been accepted heretofore, for the admission of engineering colleges to the accredited list of the Society. In formulating the standards, the Society should consider contents of engineering curricula, qualifications of engineering professors (which will necessarily involve their remunerations), engineering libraries and their use, housing, laboratory equipment, engineering research, and the achievements of engineering alumni.

2.—Require candidates for admission who are non-graduates of accredited engineering colleges, as a condition for acceptance, to pass actual examinations demonstrating educational and professional attainments equivalent to the completion of a standard engineering course. In such examinations (which should be partly written and partly oral), the candidate's professional achievement

should be given due weight. Questions for the written examinations should be prepared and the papers be graded at Society Headquarters. The examinations might be given by the Local Membership Committees.

Candidates who are graduate engineers might well be required to appear in person before the Local Membership Committees for conferences which would be somewhat of the nature of oral examinations.

3.—In my judgment, the Society might very properly and wisely proceed, in co-operation with the Society for the Promotion of Engineering Education, to formulate and secure the adoption, by accredited engineering colleges, of standard requirements for the professional engineering degree of Civil Engineer (C. E.).

Almost all our standard engineering colleges are already agreed that the professional degrees, of Civil Engineer, Mechanical Engineer (M. E.) and the like, should be awarded only after years of successful engineering practice; never for engineering college work alone, for which the standard degree is Bachelor of Science (B. S.). It is, therefore, very proper that our National Civil Engineering Society participate in deciding how much and what kind of civil engineering practice should be required before giving any person the legal right to use the title of Civil Engineer.

The present requirements for the degree of Civil Engineer vary greatly in different institutions, and I believe that the engineering colleges would, in general, welcome action by the Society which would standardize professional degrees. When so standardized, the Society might well give such degrees greater weight than at present in connection with its requirements for admission to the grade of Associate Member.

4.—The Society ought to take some active steps to aid effectively in the development of true graduate civil engineering work of high order in our engineering colleges.

It now seems settled that, at least for a considerable period in the future, the standard engineering college course will be four years, following graduation from standard secondary schools. This means that the educational attainments of the engineering graduate cannot be expected to advance markedly until that indefinite future date when the junior high school and the junior college movements develop to the point of delivering to the engineering colleges candidates for admission who have already completed one or two years of work now done in college.

The engineering colleges, however, are now actively developing one to three years of graduate engineering work, designed to serve only the best fraction of engineering graduates. The National Engineering Societies can and should do much to assist in placing such graduate work on a very high plane, and in encouraging practicing engineers to take advantage of the facilities of the graduate laboratories of our great engineering colleges.

5.—It would be well for the Society to study the feasibility and the advisability of sponsoring in the near future advanced schools, of several weeks' duration, restricted to practitioners of experience and merit, for instruction in the most advanced developments in specialized branches of civil engineering practice.

I visualize such schools as financed by fees and endowments and as gatherings where our most aspiring and promising civil engineers of ages, say, between 27 and 40, could learn at the feet of the foremost engineers, with some of the foremost scientists interpreting the latest scientific discoveries.

I dare even to dream that the Society might obtain from Congress special legal authority to conduct such schools, to award diplomas for completion of work therein, and possibly to confer upon that fraction of the membership of the Society who meet certain specified high qualifications some legal designation which will win them, in fact if not in law, some special rights and privileges in their professional practice.

AMERICAN SOCIETY OF CIVIL ENGINEERS
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PAPERS AND DISCUSSIONS

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NIAGARA POWER*

By Norman R. Gibson,† M. Am. Soc. C. E.

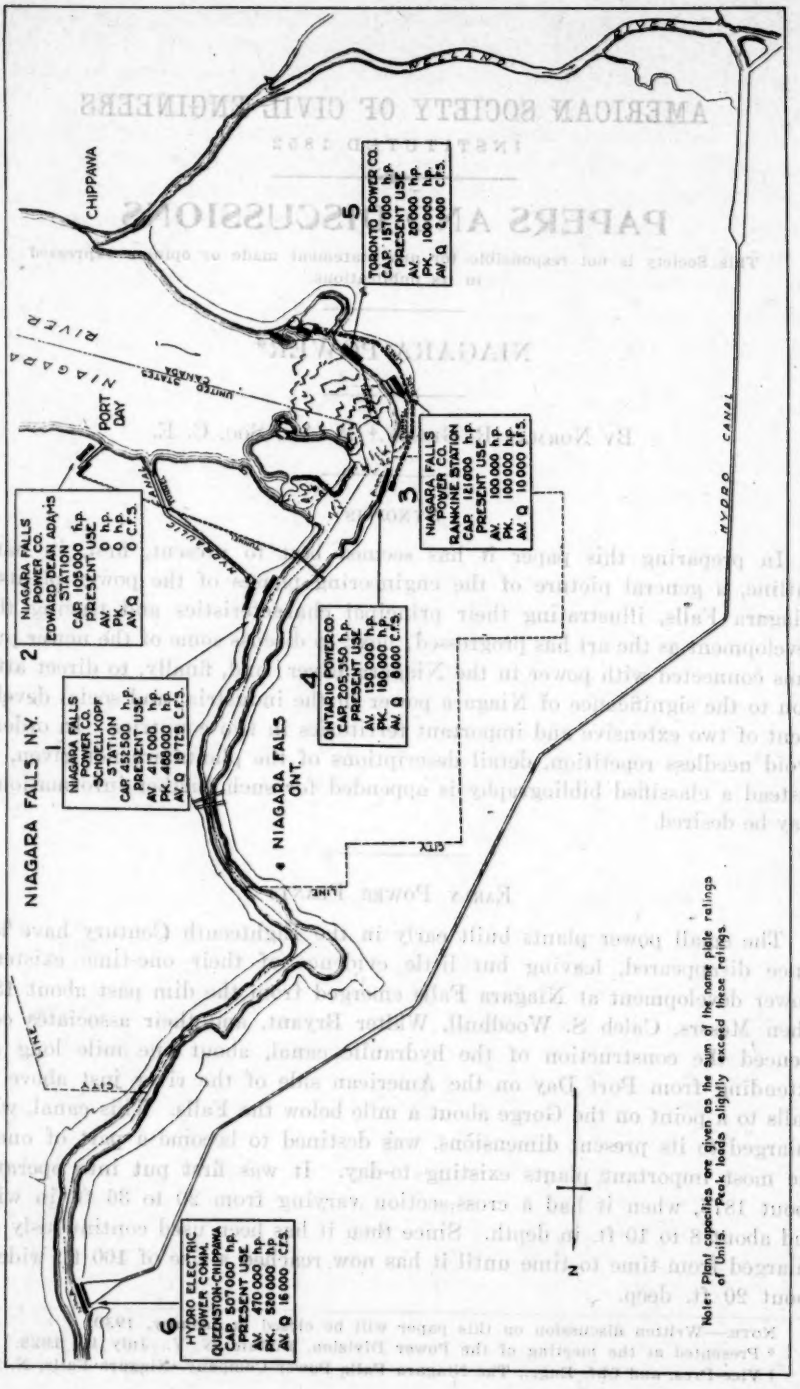
Synopsis

In preparing this paper it has seemed best to present, first, in barest outline, a general picture of the engineering phases of the power plants at Niagara Falls, illustrating their principal characteristics and tracing their development as the art has progressed; next, to discuss some of the newer problems connected with power in the Niagara River; and, finally, to direct attention to the significance of Niagara power in the industrial and social development of two extensive and important territories in two countries. In order to avoid needless repetition, detail descriptions of the plants are not given, but instead a classified bibliography is appended for such further information as may be desired.

Early Power Plants

The small power plants built early in the Eighteenth Century have long since disappeared, leaving but little evidence of their one-time existence. Power development at Niagara Falls emerged from the dim past about 1853, when Messrs. Caleb S. Woodhull, Walter Bryant, and their associates commenced the construction of the hydraulic canal, about one mile long and extending from Port Day on the American side of the river just above the Falls to a point on the Gorge about a mile below the Falls. This canal, when enlarged to its present dimensions, was destined to become a part of one of the most important plants existing to-day. It was first put into operation about 1875, when it had a cross-section varying from 20 to 36 ft. in width and about 8 to 10 ft. in depth. Since then it has been used continuously and enlarged from time to time until it has now reached a size of 100 ft. wide by about 20 ft. deep.

Note.—Written discussion on this paper will be closed in January, 1930.
* Presented at the meeting of the Power Division, Buffalo, N. Y., July 19, 1928.
† Vice-Pres. and Chf. Engr., The Niagara Falls Power Company, Niagara Falls, N. Y.

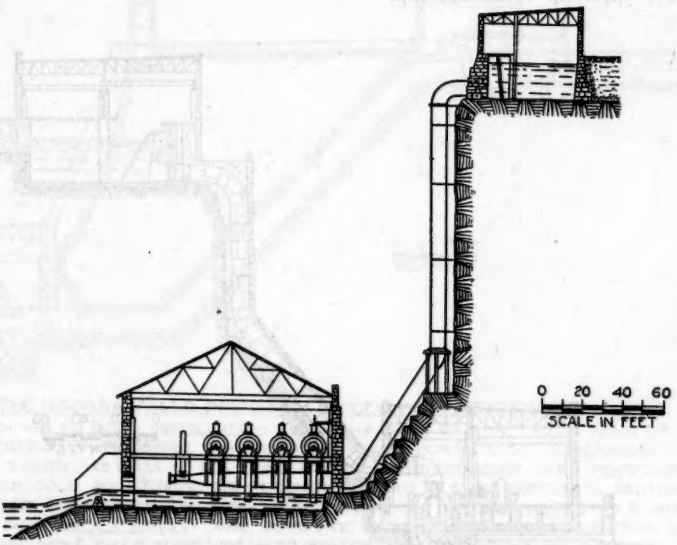


Note: Plant capacities are given as the sum of the name plate ratings of Units. Peak loads slightly exceed these ratings.

FIG. 1.—MAP SHOWING LOCATION OF EXISTING NIAGARA POWER PLANTS.

Present Power Plants

To-day (1928) there are six major hydro-electric generating stations at Niagara Falls, having a total installed capacity of 1 547 850 h.p. As shown in Fig. 1, four of these stations, aggregating an installed capacity of 990 350 h.p., are located on the Canadian side of the river, and two, with an installation of 557 500 h.p., are on the American side. The ownership and control of all these plants are vested in two organizations; the two plants on the American side and one of those in Canada being owned or controlled by The Niagara Falls Power Company, a New York corporation, and the remaining three plants on the Canadian side being under the Hydro-Electric Power Commission of Ontario.



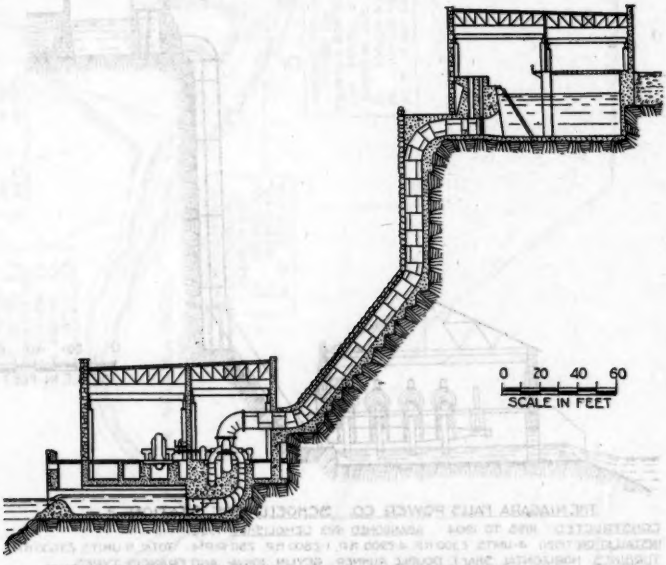
THE NIAGARA FALLS POWER CO. SCHOELLKOPF STATION 2
CONSTRUCTED: 1895 TO 1904 ABANDONED: 1923 DEMOLISHED: 1925.
INSTALLATION: (1921) 4-UNITS 2300 H.P. 4-2900 H.P. 1-2800 H.P. 250 R.P.M. TOTAL 9 UNITS, 23600 H.P.
TURBINES: HORIZONTAL SHAFT, DOUBLE RUNNER, GEYLIN, JONVAL AND FRANCIS TYPES.
GENERATORS: EARLIER INSTALLATIONS INCLUDED A.C. UNITS AT 11000 VOLT, 3 PHASE, 30 CYCLES AND 2200 VOLT SINGLE PHASE 125 CYCLES. (1921) 4-750 Kw. 300 VOLT D.C., 2-875 Kw. 180 VOLT D.C., 8-1000 Kw. 300 VOLT D.C. 4-750 Kw. 350 VOLT D.C. TWO GENERATORS DIRECTLY COUPLED TO TURBINE SHAFT. NET HEAD: ABOUT 210 FT. UNIT EFFICIENCY: 80%.
PENSTOCKS: PLATE STEEL, 1-8'-0" DIAM. 240 FT. LONG, 2-11'-0" DIAM. 270 FT. LONG. SLIDING STEEL HEAD GATES. WATER SUPPLY: CANAL 100 FT. WIDE BY 20 FT. DEEP, AND 4500 FT. LONG FROM INTAKE ABOUT ONE MILE ABOVE FALLS. SUPPLYING ALSO STATIONS 3A AND 3B.

Fig. 2.

Schoellkopf Station.—The hydraulic canal, Fig. 1, was purchased in 1877 by Jacob F. Schoellkopf and others who, a year later, organized The Niagara Falls Hydraulic Power and Manufacturing Company, using the canal to supply water to wheels in mills located at the top of the high bank along the Gorge. The water-wheels were placed in the bottom of shafts or pits sunk beneath the mills at various depths, and utilized from 25 to 86 ft. of the total 215 ft. available head.

Following early experimental stations from 1880 onward, The Niagara Falls Hydraulic Power and Manufacturing Company constructed, in 1881, its first commercial hydro-electric generating station, which was known as Station No. 1. It had a capacity of 2 000 h.p. generated by direct-current dynamos operated by rope drives from water-wheels. This station was abandoned in 1904.

Station No. 2 of this Company (Fig. 2) was begun in 1895, completed about 1904, abandoned in 1921, and demolished in 1925. Originally, it had a total of fifteen hydro-electric units with an aggregate capacity of 33 900 h.p. These units were supplied with water from the hydraulic canal by three steel-plate penstocks, each of which fed from four to six units. In 1914, six units and one penstock were abandoned, leaving nine units having an aggregate capacity of 23 600 h.p.



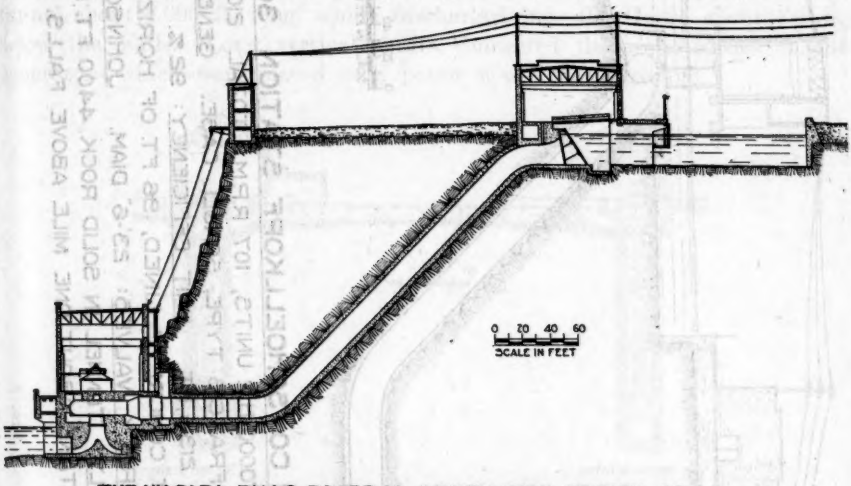
THE NIAGARA FALLS POWER CO. SCHOELLKOPF STATION 3A
CONSTRUCTED: 1905 TO 1914. INSTALLATION: 13-10000 H.P. UNITS, 300 R.P.M., TOTAL 130000 H.P.
TURBINES: HORIZONTAL SHAFT, DOUBLE RUNNER, FRANCIS TYPE, SCROLL CASE.
GENERATORS: 8000 KVA, 12000 VOLTS, 3 PHASE, 25 CYCLES. NET HEAD: 210 FT. UNIT EFFICIENCY: 86%.
PENSTOCKS: PLATE STEEL, 260 FT LONG, 9'-0" DIAM, GATE VALVES. WATER SUPPLY: CANAL 100 FT. WIDE BY 20 FT. DEEP, 4500 FT. LONG FROM INTAKE ABOUT ONE MILE ABOVE FALLS, SUPPLYING ALSO STATION 3B.

FIG. 3.

About 1903, The Niagara Falls Hydraulic Power and Manufacturing Company began the construction of Section A of its Station No. 3 (Fig. 3), to contain thirteen 10 000-h.p. units. This, together with the addition of Section B (Fig. 4) having three 37 500-h.p. units constructed during the years 1918 to 1920, and Section C (Fig. 5) with three 70 000-h.p. units constructed in the period 1922 to 1924, constitutes the largest and most important plant on the American side to-day. Its aggregate installed capacity is 452 500 h.p.

The hydraulic canal which, with frequent enlargements, had been able to supply water to all the plants of this Company, reached the limit of its capacity after the addition of Station No. 3-B. A tunnel, 32 ft. in diameter, from Port Day to the Canal Basin, was constructed, therefore, as a part of the Station No. 3-C addition. In 1927, Station No. 3 was named Schoellkopf Station in honor of Jacob F. Schoellkopf and his family, three generations of which have devoted themselves to the task of developing Niagara power.

In 1910, The Niagara Falls Hydraulic Power and Manufacturing Company became the Hydraulic Power Company of Niagara Falls, and, in 1918, the latter was consolidated with The Niagara Falls Power Company.



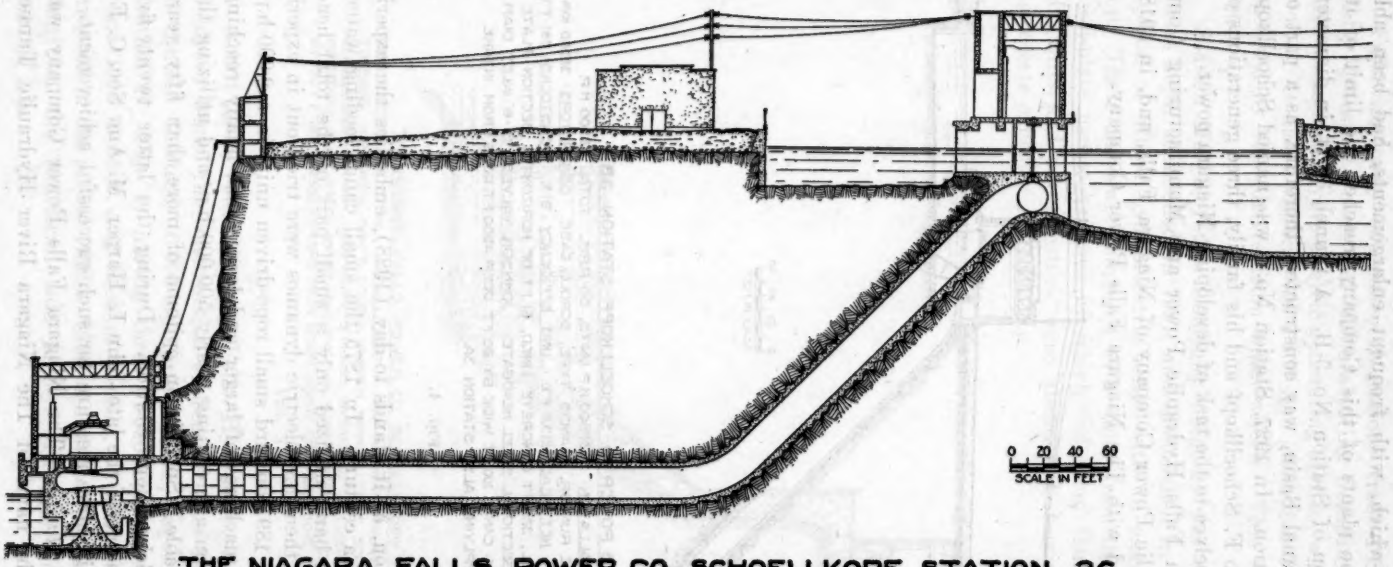
THE NIAGARA FALLS POWER CO. SCHOELLKOPF STATION 3B

CONSTRUCTED: 1918 TO 1920. INSTALLATION: 3-37500 H.P. UNITS, 150 R.P.M., TOTAL 112500 H.P. TURBINES: VERTICAL SHAFT, SINGLE RUNNER, FRANCIS TYPE, SCROLL CASE. GENERATORS: 32500 KVA. 12000 VOLTS, 3 PHASE, 25 CYCLE. NET HEAD: 213.5 FT. UNIT EFFICIENCY: 91%. PENSTOCKS: 381 FT. LONG, 15'-6" DIAM., SOLID ROCK TUNNEL, 320 FT. CONCRETE LINED, 61 FT. OF HORIZONTAL SECTION PLATE STEEL ENCASED IN CONCRETE. SECTIONAL STEEL HEADGATE. JOHNSON VALVES: 15'-6" INLET DIAM., 10'-6" OUTLET DIAM. WATER SUPPLY: CANAL 100 FT. WIDE BY 20 FT. DEEP 4500 FT. LONG FROM INTAKE ABOUT ONE MILE ABOVE FALLS, SUPPLYING ALSO STATION 3A.

FIG. 4.

The Schoellkopf Station, as it stands to-day (1928), embodies the experience and progress of half a century. In 1875 the small canal feeding waterwheels in mills along the bank utilized only a small part of the total head available. After 1880 the then new electric dynamos were tried out in experimental stations, and from 1891 onward small rope-driven units in a 2 000-h.p. station were followed by machines of larger and larger size, finally reaching 70 000 h.p. each in a station aggregating nearly 500 000 h.p. and utilizing the full 215 ft. of head available. Such is an epitome of more than fifty years of development in the Schoellkopf Station. During the latter twenty-five years of that period, the name of the late John L. Harper, M. Am. Soc. C. E., stands out as responsible in large measure for such successful achievement.

Edward Dean Adams Station.—The Niagara Falls Power Company was created in 1886, under the name of The Niagara River Hydraulic Tunnel



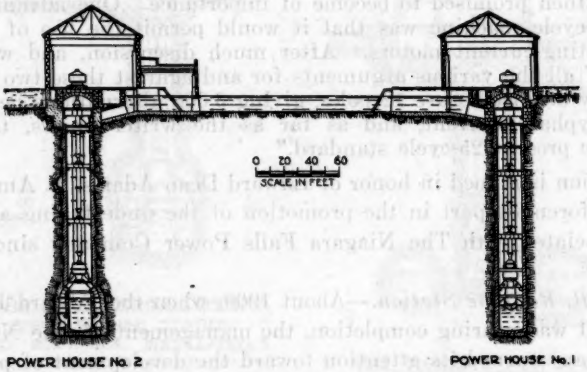
THE NIAGARA FALLS POWER CO. SCHOELLKOPF STATION 3C

CONSTRUCTED: 1921 TO 1924. INSTALLATION: 3-70000 H.P. UNITS, 107 R.P.M. TOTAL 210000 H.P. TURBINES: VERTICAL SHAFT, SINGLE RUNNER, FRANCIS TYPE, SCROLL CASE. GENERATORS: 65000 KVA. 12000 VOLTS, 3 PHASE, 25 CYCLE. NET HEAD: 213.5 FT. UNIT EFFICIENCY: 92%. PENSTOCKS: 600 FT. LONG, 21'-0" DIAM., SOLID ROCK TUNNEL, 504 FT. CONCRETE LINED, 96 FT. OF HORIZONTAL SECTION PLATE STEEL ENCASED IN CONCRETE. BUTTERFLY VALVES: 23'-6" DIAM. JOHNSON VALVES: 21'-0" INLET DIAM., 14'-0" OUTLET DIAM. WATER SUPPLY: TUNNEL IN SOLID ROCK 4400 FT. LONG, 32'-0" DIAM. HORSESHOE SHAPE, CONCRETE LINED, FROM INTAKE ABOUT ONE MILE ABOVE FALLS. TUNNEL AND CANAL INTERCONNECTED.

FIG. 5.

Power and Sewer Company, the name being changed in 1889. At that time it was debated whether the Company should develop hydraulic power to be delivered to mills located along its waterways, or pneumatic power for transmission to industrial districts and even to Buffalo, N. Y. This question was settled by the rapid development of electrical power which definitely displaced both the other proposals.

After much consideration a new type of plant was designed for the Edward Dean Adams Station. In this the water-wheels were placed in the bottom of a wheel-pit, about 24 ft. wide and 130 ft. deep, excavated in the solid rock. The discharge from the wheels was carried to the river through a flow-line tunnel about 7 000 ft. long, which discharged into the Gorge about ½ mile below the Falls. Long vertical shafts connected the water-wheels to the generators which were placed in a power house above ground.



THE NIAGARA FALLS POWER CO. EDWARD DEAN ADAMS STATION.
POWER HOUSE No. 1
CONSTRUCTED: 1890 TO 1900. TURBINES REMODELED 1910 TO 1912.
INSTALLATION: 10-5000 H.P. UNITS, 250 R.P.M., TOTAL 50000 H.P.
TURBINES: VERTICAL SHAFT, SINGLE RUNNER, FRANCIS TYPE. CYLINDRICAL GATES. GENERATORS: 5-4400 KVA. 12000 VOLT 3 PHASE, 25 CYCLES. 5-4400 KVA. 2200 VOLT 2 PHASE 25 CYCLES
NET HEAD: 135 FT. UNIT EFFICIENCY: 78% SHAFT: TUBULAR 3'-2" DIAM. PENSTOCKS: PLATE STEEL 7'-6" DIAM, 144 FT. LONG, SLIDING STEEL HEAD GATES. WATER SUPPLY: INTAKE CANAL ABOUT ONE AND A QUARTER MILES ABOVE FALLS, WIDTH 119 FT. TO 194 FT. AVERAGE DEPTH 12 FT., 1200 FT. LONG. SUPPLIES POWER HOUSE No 2 ALSO. DISCHARGE TUNNEL: HORSESHOE SHAPE, BRICK LINED, 21 FT. HIGH BY 18'-10" WIDE, 7481 FT. LONG. SAME TUNNEL DISCHARGES WATER FROM POWER HOUSE No. 2.
POWER HOUSE No. 2
CONSTRUCTED 1900 TO 1904 INSTALLATION: 11-5000 H.P. UNITS, 250 R.P.M. TOTAL 55000 H.P. TURBINES: VERTICAL SHAFT, SINGLE RUNNER, FRANCIS TYPE, CYLINDRICAL GATES. GENERATORS: 11-4400 KVA. 2200 VOLT 2 PHASE 25 CYCLES NET HEAD: 135 FT. UNIT EFFICIENCY: 73%
SHAFT: TUBULAR 3'-2" DIAM. PENSTOCKS: PLATE STEEL 7'-6" DIAM, 148 FT. LONG, SLIDING STEEL HEAD GATES. WATER SUPPLY AND DISCHARGE TUNNEL SAME AS POWER HOUSE No. 1

FIG. 6.

By 1895, alternating-current electrical power was being generated in Station No. 1 and delivered in Niagara Falls. Power was sent to Tonawanda, N. Y., and Buffalo in 1896 by means of 11 000-volt transmission. By 1900, Station No. 1 had been completed with ten units of 5 000 h.p. each, and, by

1904, Station No. 2 had been completed with a further installation of eleven units of 5 000 h.p. each. Fig. 6 shows a typical cross-section of both stations. On the completion of Section C of the Schoellkopf Station in 1924, the twenty-one units aggregating 105 000 h.p. in the Adams Station were placed in reserve.

One of the questions most frequently asked is, "What led to the adoption of twenty-five cycles as the standard frequency of alternating current generated at Niagara Falls?" This question is answered by B. G. Lamme,* as follows:

"A speed of 250 revolutions per minute was decided upon. The engineers of the power company proposed eight-pole machines, giving 2 000 alternations per minute or $16\frac{2}{3}$ cycles per second. The Westinghouse Company proposed, as an alternative, 16 poles giving $33\frac{1}{3}$ cycles, the advantage claimed for this frequency being that it was better suited for motors and rotary converters, which were then promised to become of importance. One advantage claimed for the $16\frac{2}{3}$-cycle machine was that it would permit the use of commutator type alternating-current motors. After much discussion, and weighing and balancing of all the various arguments for and against these two frequencies, it was finally decided to use 12 poles, giving 3 000 alternations per minute, or 25-cycle polyphase current, and as far as the writer knows, this was the origin of the present 25-cycle standard."

This station is named in honor of Edward Dean Adams, F. Am. Soc. C. E., who took a foremost part in the promotion of the undertaking and has been actively associated with The Niagara Falls Power Company since its inception.

William B. Rankine Station.—About 1900, when the Edward Dean Adams Station No. 1 was nearing completion, the management of The Niagara Falls Power Company turned its attention toward the development of power on the Canadian side of the river, for which it had obtained rights from the Government of the Province of Ontario in 1892. By this time sufficient experience had been gained to warrant the construction of hydro-electric generating units of 10 000 h.p. capacity. The wheel-pit and tail-race tunnel type of plant was again adopted in the design of this station (Fig. 7). By 1905, power was being delivered from this Canadian plant and by 1917, ten units had been installed having an aggregate capacity of 109 000 h.p. In 1924, Unit No. 11, of 12 000 h.p. capacity, was added, thus bringing the total installed capacity of the station up to 121 000 h.p.

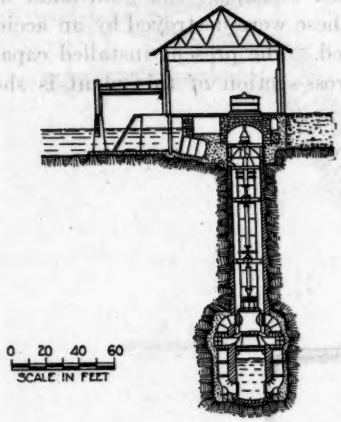
This station is named in honor of William B. Rankine, who perhaps more than any other person bore the burden of the pioneer work in The Niagara Falls Power Company.

International Railway Company.—A small installation on the Canadian side of the river, a short distance above the Horseshoe Falls, was begun in 1892 and completed with a total installation of 3 600 h.p. in 1905. This plant is owned by the International Railway Company and is used to generate direct current for the operation of the railway.

Toronto Power Company.—Power projects at Niagara Falls began to multiply early in the Twentieth Century and two more plants were con-

* "Electrical Engineering Papers," p. 602.

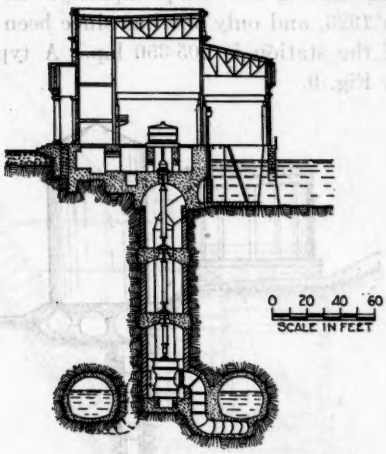
structed on the Canadian side of the river. One of these was developed by the Electrical Development Company, now commonly referred to as the Toronto Power Company, a confusion which originated by substituting the name of the transmission company for that of the developing company. This Company followed somewhat the same lines of development as were used for the Adams and Rankine Stations, but bringing the discharge end of its tail-race tunnel through the cliff directly behind the sheet of water falling over the Horseshoe Falls.



THE NIAGARA FALLS POWER COMPANY
WILLIAM BIRCH RANKINE STATION

CONSTRUCTED: 1901 TO 1924. INSTALLATION: 5 UNITS 10250 H.P., 2-12750 H.P., 3-10750 H.P., 112000 H.P., 250 R.P.M., TOTAL 11 UNITS 121000 H.P. TURBINES: VERTICAL SHAFT, FRANCIS TYPE, 7 DOUBLE RUNNER AND 4 SINGLE RUNNER, CYLINDRICAL GATES IN ALL BUT 12000 H.P. UNIT WHICH HAS WICKET GATES. GENERATORS: 5-8800 KVA., 5-10400 KVA., 1-11440 KVA., 12000 VOLT, 3 PHASE, 25 CYCLES. NET HEAD: 135 FT. UNIT EFFICIENCY: 70%. SHAFT: TUBULAR 3'-3½" DIAM. PENSTOCKS: PLATE STEEL 10'-2½" DIAM., 136 FT. LONG, SLIDING STEEL HEAD GATES. WATER SUPPLY: INTAKE CANAL ABOUT ONE QUARTER MILE ABOVE FALLS, 282 FT. TO 546 FT. WIDE, AVERAGE DEPTH 14'-6", LENGTH 300 FT. DISCHARGE TUNNEL: HORSESHOE SHAPE, BRICK LINED, 25 FT. HIGH BY 18'-10" WIDE, 2165 FT. LONG.

FIG. 7.



THE TORONTO POWER COMPANY

CONSTRUCTED: 1903 TO 1915. INSTALLATION: 7 15000 H.P. UNITS, 4 13000 H.P. UNITS, 250 R.P.M. TOTAL 11 UNITS 157000 H.P. TURBINES: VERTICAL SHAFT, DOUBLE RUNNER, FRANCIS TYPE, 250 R.P.M. GENERATORS: 7-10000 KVA., 4-8000 KVA., 12000 VOLTS, 3 PHASE, 25 CYCLES. NET HEAD: 135 FT. SHAFT: TUBULAR 2'-6" DIAM. PENSTOCKS: PLATE STEEL 10'-6" DIAM. 145 FT. LONG, SLIDING STEEL HEAD GATE. WATER SUPPLY: INTAKE AND POWER HOUSE ABOUT ONE HALF MILE ABOVE FALLS. DISCHARGE TUNNEL: 2 BRANCH TUNNELS 25 FT. HIGH AND 16 FT. TO 28'-9" WIDE, ABOUT 500 FT. LONG UNITE AND FORM MAIN HORSESHOE SHAPE TUNNEL 26 FT. HIGH BY 23.5 FT. WIDE, 1935 FT. LONG, BRICK LINED.

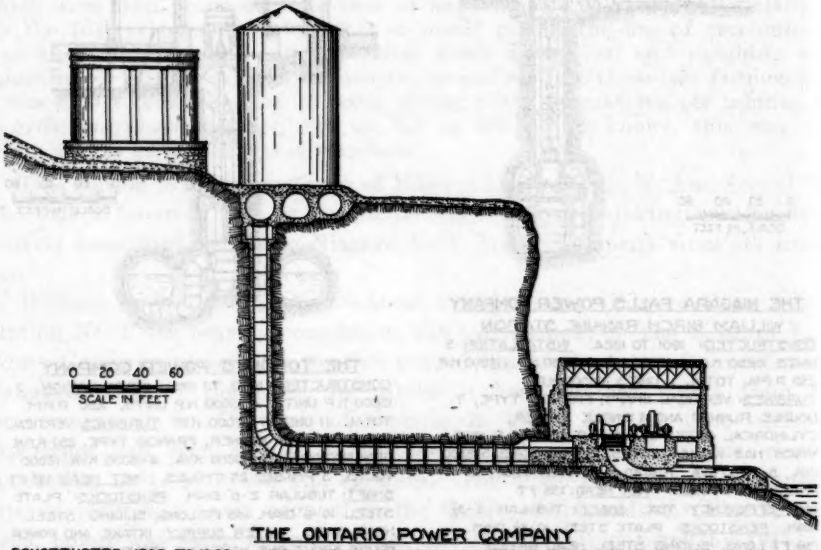
FIG. 8.

The station was first put into operation in 1906, and was completed by 1915. There are four units of about 13 000 h.p. capacity each, and seven of 15 000 h.p. each, making up a total of 157 000 h.p. for the station. A typical cross-section is shown in Fig. 8. The plant was sold to the Hydro-Electric Power Commission of Ontario in 1918.

Ontario Power Company of Niagara Falls.—The other Company to begin work early in the Twentieth Century was the Ontario Power Company. This Company departed from the types of plant previously built and used pressure conduits about 6 000 ft. long to carry water taken from the river above the

upper rapids to penstocks connected to water turbines located in a power house in the Gorge just below the Horseshoe Falls.

Power from this plant was first produced in 1905. By 1913, two main conduits, one of plate steel and one of reinforced concrete, each having an internal diameter of 18 ft., and a power house containing fourteen units having a total installed capacity of 186 050 h.p., had been completed. In 1917 this Company was purchased by the Hydro-Electric Power Commission of Ontario. In 1918, as a World War measure, a third conduit, 13½ ft. in diameter, was constructed of wood and used to supply two additional units aggregating 36 000 h.p. capacity. Both of these were destroyed by an accident in 1923, and only one has since been restored. The present installed capacity of the station is 205 350 h.p. A typical cross-section of this plant is shown in Fig. 9.



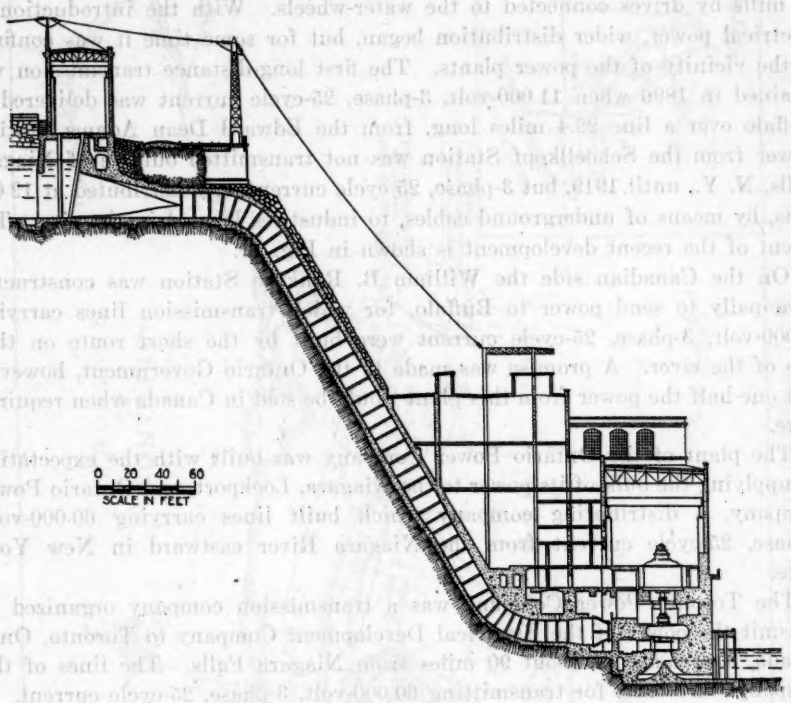
THE ONTARIO POWER COMPANY

CONSTRUCTED: 1902 TO 1923. INSTALLATION: 3-12350 H.P UNITS, 4-13250 H.P. UNITS, 4-13500 H.P. UNITS, 3-14000 H.P. UNITS, AND 1-19300 H.P. UNIT. 187½ R.P.M. TOTAL 15 UNITS 205350 H.P. TURBINES: HORIZONTAL SHAFT, 2 TURBINES ON SHAFT, FRANCIS TYPE, SCROLL CASE, WICKET GATES. GENERATORS: 3-7500 KVA., 4-8773 KVA., 7-8776 KVA., 1-15000 KVA., 12000 VOLTS, 3 PHASE, 25 CYCLES. NET HEAD: 180 FT. PENSTOCKS: PLATE STEEL VARYING FROM 225 FT TO 307 FT LONG, 14 - 3'-0" DIAM. AND 1 - 10'-6" DIAM., GATE VALVES 12 UNITS AND JOHNSON VALVES 3 UNITS. WATER SUPPLY: INTAKE ABOUT THREE QUARTERS OF A MILE ABOVE FALLS. 3 PIPE LINES, 1 PLATE STEEL, 6300 FT. LONG, 18'-0" DIAM., 1 REINFORCED CONCRETE, 6500 FT. LONG, 18'-0" DIAM., 1 WOOD STAVE, 6700 FT LONG, 13'-6" DIAM. SIMPLE OVERFLOW CHAMBER AT END OF STEEL PIPE LINE AND DIFFERENTIAL SURGE TANKS AT END OF CONCRETE AND WOOD STAVE PIPE LINES.

FIG. 9.

Queenston-Chippawa Development.—The latest power plant built at Niagara Falls was constructed on the Canadian side during the years 1917 to 1921, by the Hydro-Electric Power Commission of Ontario. This plant takes its water from the Niagara River through an intake at the mouth of the Welland River and by reversing the natural flow for a distance of 4½ miles the water is delivered to a canal, 48 ft. wide and about 30 ft. deep, below the

water-line. The canal was excavated in earth and solid rock for a distance of 8 miles to a point on the Gorge of the Lower Niagara River about 2 miles above the Village of Queenston, Ont., Canada.



THE HYDRO-ELECTRIC POWER COMMISSION OF ONTARIO
QUEENSTON - CHIPPAWA DEVELOPMENT

CONSTRUCTED: 1917 TO 1925. INSTALLATION: 5-55000 H.P. UNITS AND 4-58000 H.P. UNITS, 187.5 R.P.M., TOTAL 9 UNITS 507000 H.P. TURBINES: VERTICAL SHAFT, SINGLE RUNNER, FRANCIS TYPE, SCROLL CASE. GENERATORS: 5-45000 KVA., 4-55000 KVA., 12000 VOLTS, 3 PHASE, 25 CYCLES. NET HEAD: 305 FT. PENSTOCKS: 437 FT. LONG, 65 FT. OF UPPER HORIZONTAL SECTION SOLID ROCK CONCRETE LINED, 372 FT. PLATE STEEL ENCASED IN CONCRETE, 5 ARE 18'-0" DIAM. FOR ABOUT 2/3 THEIR LENGTH AND THEN REDUCE TO 14'-0" DIAM., AND 4 ARE 16'-0 DIAM. THROUGHOUT, SLIDING STEEL HEAD GATES. JOHNSON VALVES: 14'-0" INLET DIAM., 10'-0" OUTLET DIAM. WATER SUPPLY: INTAKE FROM NIAGARA RIVER AT MOUTH OF WELLAND RIVER, FLOW IN WHICH IS REVERSED FOR A DISTANCE OF 4 1/2 MILES TO CANAL ENTRANCE. CANAL 8 MILES LONG. EXCAVATION IN EARTH AND SOLID ROCK. ROCK SECTION 48 FT. WIDE, 30 FT. DEEP, CONCRETE LINED.

FIG. 10.

The gross head from intake to tail-water is 312 ft. Operation was commenced in December, 1921, and, by 1925, the ninth and last unit was completed. The first five units have capacities of 55 000 h.p. each, and the last four are 58 000 h.p. each, making the total installed capacity 507 000 h.p. A typical cross-section of the generating station is shown in Fig. 10.

Transmission and Distribution

It will naturally be asked, "Where and how has Niagara power been used?" Of course, in the early stages of development all the power was used locally in mills by drives connected to the water-wheels. With the introduction of electrical power, wider distribution began, but for some time it was confined to the vicinity of the power plants. The first long-distance transmission was attained in 1896 when 11 000-volt, 3-phase, 25-cycle current was delivered in Buffalo over a line 22.4 miles long, from the Edward Dean Adams Station. Power from the Schoellkopf Station was not transmitted outside of Niagara Falls, N. Y., until 1919, but 3-phase, 25-cycle current was distributed at 12 000 volts, by means of underground cables, to industries located in the city. The extent of the recent development is shown in Fig. 11.

On the Canadian side the William B. Rankine Station was constructed principally to send power to Buffalo, for which transmission lines carrying 22 000-volt, 3-phase, 25-cycle current were built by the short route on that side of the river. A promise was made to the Ontario Government, however, that one-half the power from this plant would be sold in Canada when required there.

The plant of the Ontario Power Company was built with the expectation of supplying the bulk of its power to the Niagara, Lockport, and Ontario Power Company, a distributing company which built lines carrying 60 000-volt, 3-phase, 25-cycle current from the Niagara River eastward in New York State.

The Toronto Power Company was a transmission company organized to transmit the power of the Electrical Development Company to Toronto, Ont., Canada, a distance of about 90 miles from Niagara Falls. The lines of the Company were built for transmitting 60 000-volt, 3-phase, 25-cycle current.

About 1903, a number of municipalities in Western Ontario, noting that no plans had yet been made to send Niagara power to them, and apprehensive lest they might be left in the lurch, petitioned the Government of the Province for permission to form an organization to purchase power from the generating companies at Niagara Falls and transmit it to their municipalities. As a result the Hydro-Electric Power Commission of Ontario was created in 1905, which contracted for the supply of 100 000 h.p. from the Ontario Power Company and built transmission lines to distribute this power throughout the southwestern end of the Province. This system was built for transmitting 110 000-volt, 3-phase, 25-cycle current.

From such beginnings the present distribution (Fig. 11) was gradually developed until now (1928), by purchases and consolidations, the generation and transmission of Niagara power throughout the western part of the State of New York and the southwestern part of the Province of Ontario are largely concentrated in two large systems. One of these serves a population of more than 2 500 000 in the State of New York (and in a small part of the Province of Ontario). The territory involved has an area of 14 500 sq. miles extending to the east as far as Altmar, N. Y., a distance of 200 miles, and to the south, 100 miles to the border of the State of Pennsylvania.

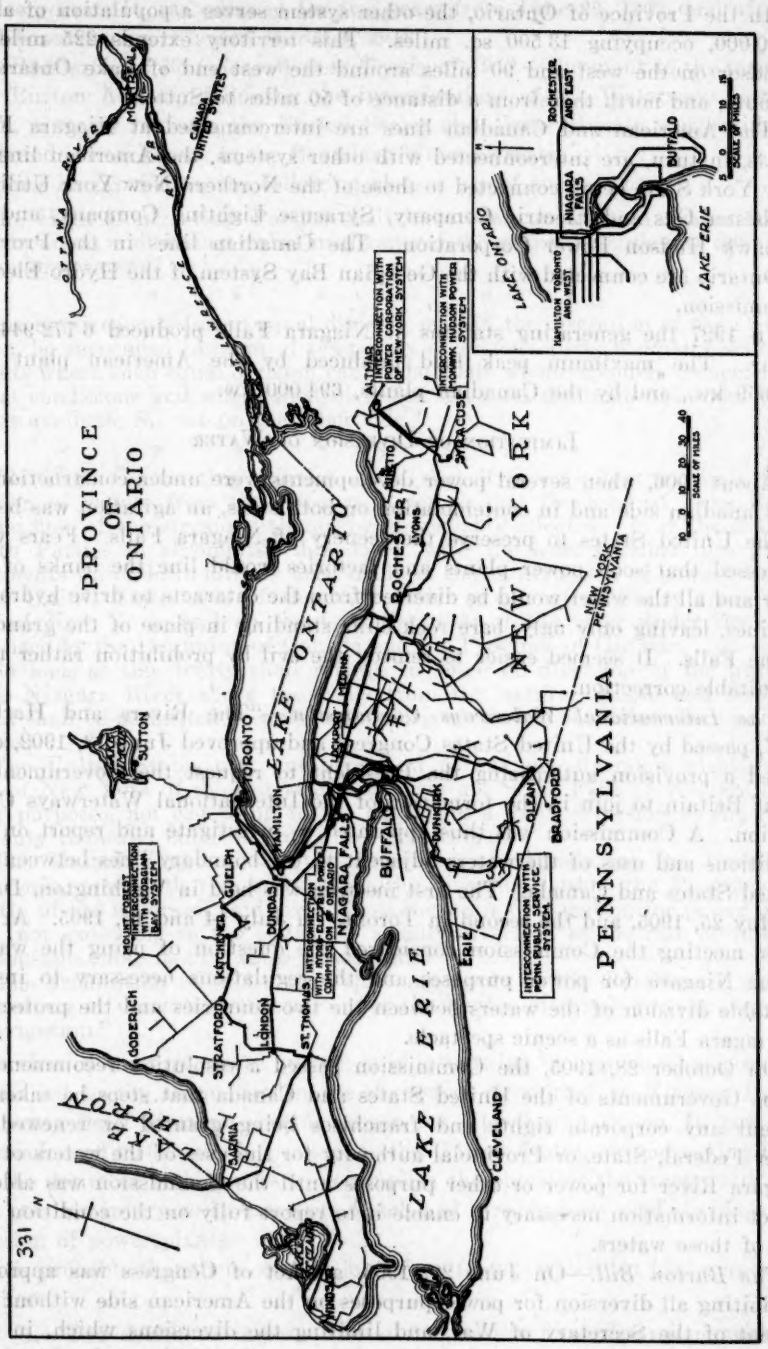


FIG. 11.—MAP OF TERRITORIES SERVED BY NIAGARA POWER.

In the Province of Ontario, the other system serves a population of about 1 500 000, occupying 13 500 sq. miles. This territory extends 225 miles to Windsor on the west and 90 miles around the west end of Lake Ontario to Toronto, and north therefrom a distance of 50 miles to Sutton.

The American and Canadian lines are interconnected at Niagara Falls. These, in turn, are interconnected with other systems, the American lines in New York State being connected to those of the Northern New York Utilities, Rochester Gas and Electric Company, Syracuse Lighting Company, and the Mohawk Hudson Power Corporation. The Canadian lines in the Province of Ontario are connected with the Georgian Bay System of the Hydro-Electric Commission.

In 1927 the generating stations at Niagara Falls produced 6 772 944 200 kw-hr. The maximum peak load produced by the American plant was 365 659 kw., and by the Canadian plants, 694 000 kw.

Limitation of Diversion of Water

About 1906, when several power developments were under construction on the Canadian side and in contemplation on both sides, an agitation was begun in the United States to preserve the scenery of Niagara Falls. Fears were expressed that soon power plants and factories would line the banks of the river and all the water would be diverted from the cataracts to drive hydraulic turbines, leaving only ugly, bare rock cliffs standing in place of the grandeur of the Falls. It seemed easier to remedy the evil by prohibition rather than by suitable correction.

The International Waterways Commission.—"The Rivers and Harbors Act", passed by the United States Congress and approved June 13, 1902, contained a provision authorizing the President to request the Government of Great Britain to join in the formation of the International Waterways Commission. A Commission was thus appointed to investigate and report on the conditions and uses of the waters adjacent to the boundary lines between the United States and Canada. The first meeting was held in Washington, D. C., on May 25, 1905, and the second in Toronto on July 14 and 15, 1905. At the latter meeting the Commission considered the question of using the waters of the Niagara for power purposes and the regulations necessary to insure equitable division of the waters between the two countries and the protection of Niagara Falls as a scenic spectacle.

On October 28, 1905, the Commission passed a resolution recommending to the Governments of the United States and Canada that steps be taken to prevent any corporate rights and franchises being granted or renewed by either Federal, State, or Provincial authority for the uses of the waters of the Niagara River for power or other purposes until the Commission was able to collect information necessary to enable it to report fully on the condition and uses of those waters.

The Burton Bill.—On June 29, 1906, an Act of Congress was approved prohibiting all diversion for power purposes on the American side without the consent of the Secretary of War, and limiting the diversions which, in any event could be permitted, to 15 600 cu. ft. per sec. This bill remained in force

for three years. It was renewed for two years from June 29, 1909, and after two other extensions it lapsed March 4, 1913.

The Intenational Boundary Waters Treaty of 1910.—Pursuant to the terms of the Burton Act and following the investigations of the Waterways Commission, a treaty between the United States and Great Britain was ratified in 1910 and is now known as the Boundary Waters Treaty.

Among its provisions, the following clauses are of special interest:

"The High Contracting Parties shall have each on its own side of the boundary equal and similar rights in the use of the waters hereinbefore defined as boundary waters."

* * * * * *

"The requirements for an equal division may in the discretion of the Commission, be suspended in cases of temporary diversions along boundary waters at points where such equal division cannot be made advantageously on account of local conditions and where such diversion does not diminish elsewhere the amount available for use on the other side."

* * * * * *

"The High Contracting Parties agree that it is expedient to limit the diversion of waters from the Niagara River so that the level of Lake Erie and the flow of the stream shall not be appreciably affected. It is the desire of both Parties to accomplish this object with the least possible injury to investments which have already been made in the construction of power plants on the United States side of the river under grants of authority from the State of New York, and on the Canadian side of the river under licenses authorized by the Dominion of Canada and the Province of Ontario.

"So long as this treaty shall remain in force no diversion of the waters of the Niagara River above the Falls from the natural course and stream thereof shall be permitted except for the purposes and to the extent hereinafter provided.

"The United States may authorize and permit the diversion within the State of New York of the waters of said river above the Falls of Niagara, for power purposes, not exceeding in the aggregate a daily diversion at the rate of twenty thousand cubic feet of water per second.

"The United Kingdom, by the Dominion of Canada, or the Province of Ontario, may authorize and permit the diversion within the Province of Ontario of the waters of said river above the Falls of Niagara, for power purposes, not exceeding in the aggregate a daily diversion at the rate of thirty-six thousand cubic feet of water per second.

"The prohibition of this article shall not apply to the diversion of water for sanitary or domestic purposes, or for the service of canals for the purpose of navigation."

In view of the first clause quoted, it is interesting to inquire what possible reasons led to the inequality in the diversions permitted at Niagara Falls. The treaty itself gives no specific reason for allowing 36 000 cu. ft. per sec. to Canada and 20 000 cu. ft. per sec. to the United States, beyond saying that it is the desire of both parties to accomplish the object of the treaty with the least possible injury to investments which had already been made in the construction of power plants.

It is probable that there were at least four factors which led to the unequal division:

1.—The desire to impose limitations on the total diversions without injury to investments already made.

2.—The equalizing effect of generating power on the Canadian side and transmitting it to the United States.

3.—The position of two of the plants on the Canadian side, with intakes below the crest of the upper cascades and, therefore, not affecting the American Fall.

4.—The works constructed at Chicago, Ill., for the diversion of water from the Great Lakes.

The limitation of diversion of water imposed by the treaty restricts the aggregate output of the power plants at Niagara Falls to an amount considerably below their combined installed capacities. It is for this reason that the Edward Dean Adams Plant is held in reserve as previously stated.

Table 1 shows power statistics for the various plants when operated in accordance with the provisions of the treaty.

TABLE 1.—LIMITS OF OPERATION, NIAGARA POWER PLANTS.

Plant.	Installed capacity, in horse-power.	PRESENT USE UNDER TREATY LIMITATIONS.	
		Average horse-power.	Peak horse-power.
Schoellkopf	452 500	417 000	488 000
Edward Dean Adams	105 000	0	0
William B. Rankine	121 000	100 000	100 000
International Railway Company	3 600	.....	.....
Ontario Power Company	205 350	130 000	180 000
Toronto Power Company	157 000	20 000	100 000
Queenston-Chippawa	507 000	470 000	520 000
Total	1 551 450	1 137 000	1 388 000

EFFECT OF POWER DEVELOPMENT ON LAKE ERIE AND THE NIAGARA RIVER

Relation Between Lake Erie Stage and Niagara Discharge.—The equation relating the discharge of the Niagara River to the stage of Lake Erie has been determined by the engineers of the United States Government as follows:

$$Q = 3\,904\,(H - 558.37)^{\frac{3}{2}} \quad \ldots\ldots\ldots\ldots\ldots\ldots (1)$$

in which,

Q = discharge of the river, in cubic feet per second; and,

H = elevation above mean sea level of the surface of the water in the lake, Buffalo Gauge.

The mean value of H for the period, 1860 to 1927, inclusive, was 572.50.

Considering the relation of Equation (1) roughly, it may be stated that, when the stage of Lake Erie has risen 1 ft. above this mean level, there will be an increase of about 22 400 cu. ft. per sec. in the rate of discharge of the Niagara River; and, when the stage of the lake has fallen 1 ft. below its mean level, there will be a corresponding reduction in discharge of about 21 600 cu. ft. per sec.

Variations in Lake Levels and Discharge.—There are three kinds of variations in lake level and discharge of the Niagara River:

(1) Seasonal variations every year.
(2) Cycles of change due to variable supply from precipitation and run-off.
(3) Daily fluctuations on account of the changes in atmospheric pressure and in the direction and velocity of winds.

A study of the records of the monthly mean lake levels and discharges of the Niagara River will show that the normal seasonal variation in lake level is about 1.5 ft., and that the normal seasonal variation in discharge of the Niagara River is about 31 900 cu. ft. per sec.

In addition to the seasonal variations there are periodic fluctuations in lake level and corresponding variations in river discharge extending over a period varying from seven to eleven years, the effect of which is to make the difference between the maximum mean monthly level of Lake Erie and its minimum mean monthly level about 5.0 ft. These variations correspond to a difference between the maximum and the minimum discharges of the Niagara River of about 107 900 cu. ft. per sec.

Changes in the direction and velocity of winds on Lake Erie and in atmospheric pressure produce greater fluctuations, than the foregoing and independent of them, but only for periods of short duration. The difference between the maximum and the minimum level of the lake from this cause is about 13.0 ft. Table 2 shows the ranges in level and the corresponding discharges for the period, 1860 to 1927, inclusive.

TABLE 2.—VARIATIONS OF LEVEL IN LAKE ERIE AT BUFFALO, N. Y., AND OF DISCHARGE OF THE NIAGARA RIVER.

	Elevation, in feet.	Discharge, in cubic feet per second.
Average monthly mean (1860-1927 inclusive)	572.50	207 400
Average maximum monthly mean	573.18	222 500
Average minimum monthly mean	571.73	190 600
Maximum monthly mean (June, 1876)	574.68	257 200
Minimum monthly mean (February, 1926)	569 72	149 300
Maximum lake level due to wind	580.28	.....
Minimum lake level due to wind	567.38	.....

Effect of Diversions.—It is claimed that the navigable depths of Lake Erie have been rendered insufficient from various causes, such as:

(1) Deficient precipitation and run-off.
(2) Diversions of water from the Great Lakes through:
  a.—Chicago Drainage Canal
  b.—Welland Canal
  c.—Black Rock Ship Canal
  d.—New York State Barge Canal
  e.—Power plants at Niagara Falls.

Prior to 1927, the most authoritative estimate of the amount of the lowering at mean stage caused by all these diversions was 0.76 ft.* Of this total,

* Report on the Diversion of Water from the Great Lakes and Niagara River, by J. G. Warren, M. Am. Soc. C. E., p. 375.

the amount of lowering caused by diversions for power purposes at Niagara Falls was considered to be 0.1 ft., at least one-half of which had been compensated for by changes in the bed of the river. The amount of this lowering has been placed by other estimates as high as 3½ in.

Actually, however, measurements made in the Niagara River during the summer of 1925, under the direction of the District Engineer Office, at Buffalo, indicate that there has been no appreciable change in the relation between the discharge of the river and the stage of Lake Erie since 1900 when measurements were made by the U. S. Lake Survey Office under the direction of Francis C. Shenehon, M. Am. Soc. C. E. Such differences as were found between the measurements of 1925 and 1900 were in the opposite direction, indicating that the level of Lake Erie for a given discharge of the Niagara River had become slightly higher than in 1900. This is due probably to compensatory changes in the shore line and bed of the river, which have offset any effects that otherwise might have been caused by diversion.

Back-water studies of the flow of the Niagara River indicate that it would require a lowering of approximately 2.0 ft. in the river level at Port Day to cause a decrease in the level of Lake Erie by as much as 3.5 in. No such lowering at Port Day has occurred.

Proposed Regulation.—It has been proposed from time to time to hold Lake Erie, as well as the other Great Lakes, as nearly as possible at a uniform elevation, in order that the navigable depth will not be lessened during the low-water season each year, particularly in years of low levels following periods of deficient natural supply. For this purpose various means have been suggested, such, for instance, as a regulating dam with gates and sluices at the outlet of Lake Erie, or across the Niagara River between Port Day and Chippawa, Ont., Canada. In operating such a dam it is proposed to raise or lower the gates so as to control the outflow from the lake, allowing more water to flow out when the lake is rising and less when it is falling. It is not usually expected by this means to hold it at a uniform level, but merely to restrict the range of variation to something less than when it is left free under natural conditions.

Evidently the monthly mean level of the lake cannot be kept more nearly uniform without at the same time causing more variation in the monthly mean flow of the Niagara River, assuming that the supply to the lake will vary from natural causes as in the past. In periods of deficient supply a part of the flow of the river comes from the water stored in the lake, thereby causing lake levels to fall; and in periods of excess supply the water cannot flow through the river as fast as it enters the lake, thus causing lake levels to rise.

If Lake Erie is to be regulated in such a way that the difference between the maximum and the minimum mean monthly water level will be less than in the past, the effective storage capacity of the lake will be reduced. Then the difference between the volumes of water discharged in the Niagara River at rates higher and lower than the mean must necessarily be greater than in the past. Conversely, if the flow of the river is to be made more nearly uniform, a greater storage capacity will be required; and, consequently, the difference between the maximum and the minimum level of Lake Erie must be greater than it has been in the past. If it is desired to increase the depth

of Lake Erie for navigation purposes and, at the same time, to obtain greater regularity of flow in the Niagara River, it would be necessary to provide regulating works which will have the effect of raising the mean level of the lake to a higher plane. The necessary fluctuations in lake level would then occur above and below this plane to the extent necessary to maintain the greatest possible degree of regularity in the flow of the river.

The area of Lake Erie is approximately 10 000 sq. miles. When its level is lowered 1 ft. below the mean, a volume of water is supplied to the Niagara River sufficient to maintain its entire average flow for about sixteen days. If the flow of the river were completely shut off for one day, the use of the water for power and scenic purposes would cease, but the level of the lake would be raised only about 0.75 in. It is thus apparent that obstructions which tend to interfere temporarily with the flow of the river are of much greater significance in respect to variation in flow than to water level in the lake.

Proposed Regulation and Compensation Works.—Regulating works and compensating works for Lake Erie have been proposed from time to time since about 1900. The object of regulating works is to control the outflow so as to increase the navigable depths during otherwise naturally low-water periods.

Compensating works are different in that they are designed only to raise the mean level of the lake without attempting to control the natural variations. Fluctuations will occur above and below a certain plane higher than that existing before the works were constructed; but the depths at all stages will be greater.

Location of Proposed Regulating Works.—It has been proposed that such works be located either at the entrance to the Niagara River, near the outlet of Lake Erie, or in the river, between Port Day and Chippawa. If the value of lake regulation is sufficient to offset the economic loss of the present natural conservation of water supply for power purposes in the Niagara and St. Lawrence Rivers, the former location is preferable.

A regulating dam between Port Day and Chippawa just above the Falls, would offer serious difficulties from the standpoint of power development. No structure must be placed in the Niagara River which will endanger the supply of water to the power plants by interfering with the passage of the large volumes of ice each winter. The height to which the water would have to be raised at Port Day to produce an appreciable effect on the levels of Lake Erie would be so great that important additions would have to be made to the power plants.

Lands lying along the river above Niagara Falls and along the tributary creeks, would have to be protected and drained. The sewage of La Salle, Tonawanda, and North Tonawanda, N. Y., would have to be cared for. A guard lock would have to be constructed at the entrance to the Barge Canal at Tonawanda; and it is probable that the Buffalo Breakwater and the dike forming the Black Rock Ship Canal would have to be modified.

Referring now to the economic loss of power incurred by regulating Lake Erie so that its variation in level will be less than in the past, it will

be assumed that the usable flow of the river will be some function of the "50% time flow" referred to by the Advisory Committee of the International Electrotechnical Commission of Rating of Rivers. It will be noted that the 50% time flow of the Niagara River may be utilized with an average load factor as high as 96 per cent.

One proposed rule of regulation may be applied to the measured discharge of the Niagara River during the period 1860 to 1924, inclusive, on the assumption that the minimum flow of the Niagara River would not be permitted to be less than 180 000 cu. ft. per sec. Then to obtain a regulation of mean monthly levels of Lake Erie within a range of 1.5 ft. and to provide for a diversion of 13 000 cu. ft. per sec. from and above the lake for navigation and other purposes, it would be necessary to maintain the regulated minimum flow of 180 000 cu. ft. per sec. approximately 68% of the time.

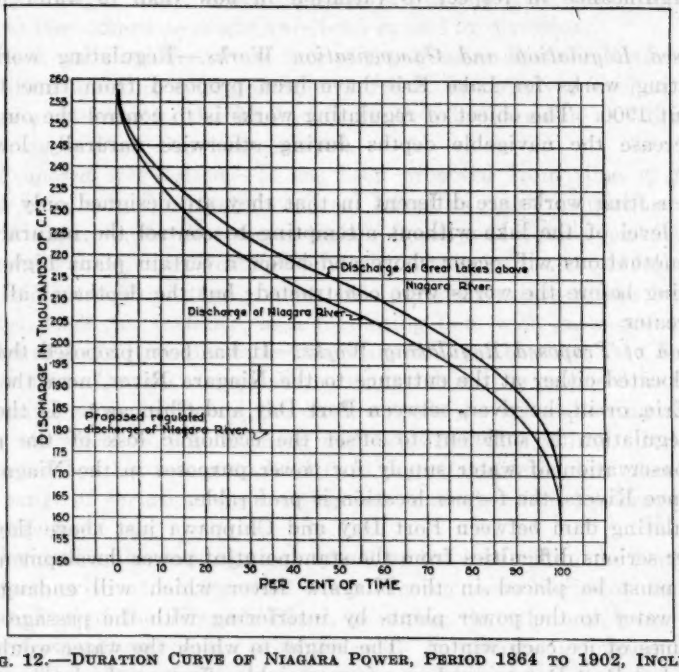


FIG. 12.—DURATION CURVE OF NIAGARA POWER, PERIOD 1864 TO 1902, INCLUSIVE.

Under natural conditions during this period the 50% time flow from the Great Lakes above the Niagara River was approximately 214 500 cu. ft. per sec. and for 3½% of the time the flow was less than 180 000 cu. ft. per sec.

Integrating under the natural and regulated duration curves (Fig. 12), it will be found that the volume of usable water added to the natural low flows by the regulation of the minimum to 180 000 cu. ft. per sec. is about 1.8% of the volume of usable water lost from the natural 50% time flow. The net loss to power development on the Niagara River is, therefore, nearly 2 000 000 000 kw.-hr. per year. To this would have to be added also the loss of power on the St. Lawrence River.

The value of this power must be considered as a charge against the benefits to be derived from regulated navigable depths and from the use of water diverted for other purposes. This conclusion is not altered by reason of the fact that part of the flow of the Niagara River is required for scenic purposes, because such water, which has been fixed by some authorities at 80 000 cu. ft. per sec., is considered to be the first claim on the river, and what remains is to be available for power purposes. The difference between the usable quantities indicated is not altered, therefore, by any such fixed allowance for scenic purposes.

Economic Benefits of Niagara Power

The question of the value of Niagara power to the districts which it serves is especially important because so much is being said about the remarkable and undeniable accomplishments in the field of steam-electric generation. It would be a mistake, however, to assume that recent great improvements in the economy of steam generation, forming as they do one of the brightest chapters in the record of the Engineering Profession, have yet reached the point, or are indeed likely to reach that point, where all water power may be thrown into the discard.

Comparisons of the cost of electric service in widely separated areas of the Continent, while of considerable interest, are perhaps of doubtful significance because charges for electrical service are so largely dependent on economic conditions in the localities where the service is rendered. Rates for electric light and power depend primarily on the capital costs of the generating, transmitting, and distributing systems, and on the number of kilowatt-hours which can be sold per dollar of capital invested. The former, in turn, depend on the physical conditions affecting the construction of the systems and on the costs of labor and materials, and the value of land. The kilowatt-hours, on the other hand, depend on the density of population and the value to the user of the service rendered. Taxes and the amounts reserved for plant renewals are important items of annual expense in addition to labor cost and interest on the invested capital.

There is a tendency to assume in public discussions of electric rates, that electricity as a commodity differs from nearly every other one in the world, in that it can be sold for the same price no matter where or under what conditions it is produced. It is also frequently assumed that electric service is a monopoly requiring something more than the ordinary economic laws to regulate its price.

Protection of the public interest by regulatory commissions does not alter the fact that in many ways the electrical business is highly competitive. Electricity competes with gas, oil, and other fuels in the same district. One district competes with another for the growth of business, and some industries largely dependent on electricity compete with like industries over the entire world. More especially, perhaps, electricity competes with all other commodities, such as motor cars, radios, etc., for its share of the consumer's dollar. Such competition effectively tends to regulate prices of electric service to their minimum economic level.

TABLE 3.—NIAGARA POWER—COST AND CAPACITY, INCLUDING AUXILIARY PLANTS, 1926.

Territory.	Fixed electrical capital.	Installed capacity, in kilowatts.	Capital per kilowatt.
Western New York.....	$205 932 420	770 989	$267
Southwestern Ontario..	$251 938 759	814 826	$309

The data in Tables 3 and 4, taken mainly from published financial reports, are presented not to indicate that rates for electricity in some other district should be different from what they are, but merely to show how the large territories served by Niagara power have benefited by the utilization of a small part of the energy made available by this great gift of Nature. Later, it will be indicated how further developments may be made without materially sacrificing those æsthetic benefits which are to be derived by the contemplation of the scenery of the Falls.

TABLE 4.—NIAGARA POWER—REVENUES, 1926.

Territory.	Electrical operating revenue from sale of electrical energy.	Electrical operating revenue from sale of electrical energy (taxes deducted).	Total kilowatt-hours generated or purchased.	REVENUE, IN MILLS PER KILOWATT-HOUR.	
				Taxes included.	Taxes deducted.
Western New York....	$41 558 864	$36 531 624	4 495 959 612	9.24	8.13
Southwestern Ontario.	26 093 911	25 804 658	2 821 791 607	9.25	9.14

The average revenue received per kilowatt-hour from the sale of power in other territories less favorably situated and served largely by steam, is from two and one-half to three times as great. These statistics prove conclusively, therefore, that the economic benefits of Niagara power have been passed on to the users. They disprove the assumption, frequently made, that the difference between the cost of water power and steam power is collected from the consumer and retained wholly for the benefit of the shareholders of the companies undertaking the development.

ULTIMATE DEVELOPMENT OF NIAGARA FALLS

Preservation of their scenic grandeur is the most important factor involved in the ultimate development of the Falls of Niagara. The discussion of this subject would be quite simple if it could be based on Lord Kelvin's prophecy, which, on the occasion of his visit to Niagara Falls in 1897, he expressed in these words:

"I look forward to the time when the whole water from Lake Erie will find its way to the lower level of Lake Ontario through machinery, doing more good for the world than even that great benefit which we now possess in contemplation of the splendid scene which we have before us in the waterfall of Niagara. I wish I could live to see this grand development. I do not hope that our children's children will ever see the Niagara cataract."

Under such conditions scenery would become a negligible factor, and power can be imagined as enthroned in two huge generating stations, extending half a mile along each side of the river at the foot of the Lower Rapids near Lewiston and Queenston, Ont. Giant generating units would produce 7 000 000 h. p. "on peak" and an average of about 6 000 000 h. p. daily throughout the year.

Such was the vision of Lord Kelvin thirty years ago. To-day, it is doubtful whether any one would venture to make such a prediction. On the contrary, for the past twenty years attention has been directed not only to the possibility of diverting more water for power purposes, but to showing how to preserve the scenery of the Falls and Rapids by providing works to remedy the effects of diversion and of the continued rapid recession of the crest of the Horseshoe Falls.

Major Keller.—Commencing with the report of Maj. Charles Keller, U. S. A. (now Colonel, U. S. A. (*Retired*)), in 1908 on the preservation of Niagara Falls, a few quotations will be given. Major Keller states:

"In my opinion, the damage already done, and that which may be anticipated from further diversions and from the impending fall in the level of Lake Erie, may be largely, if not entirely, remedied by a submerged dam placed in the bed of the river immediately above the Horseshoe Falls."

International Waterways Commission.—About the same time the International Waterways Commission, in its joint report dated 1908, made the following recommendation with reference to diversions around the Lower Rapids:

"It is our opinion that about 40 000 cubic feet per second can be diverted without perceptible injury to the rapids, * * *."

John L. Harper.—Following these two recommendations, the late John L. Harper, M. Am. Soc. C. E., in 1918, published a pamphlet directing attention to the need for the preservation of scenic effects, stating:

"* * * it should be the policy of those controlling the falls at Niagara to have constructed in the bed of the river, above the Horseshoe Falls, invisible current deflectors which would make impossible the gathering of the whole river into a deep narrow gorge, and would again deflect its waters over the sides and heels of a re-established horseshoe * * * causing the whole contour of the fall to wear uniformly * * *."

Colonel J. G. Warren.—One year afterward, the well-known Warren report of Col. J. G. Warren, U. S. A., M. Am. Soc. C. E., Division Engineer at Buffalo, contained the following:

"If the remedial works * * * are provided, it is believed a total diversion of 80 000 cubic feet per second may be made around the Falls, and 40 000 around the Whirlpool and Lower Rapids without injury to the scenic beauty, and without endangering the ice discharging capacity of the Falls or Rapids, these diversions to be divided equally between Canada and the United States."

Board of Engineers for Rivers and Harbors.—This report was then submitted to the Board of Engineers for Rivers and Harbors, headed by Brig. Gen. H. Taylor, U. S. A., M. Am. Soc. C. E. (now Major-General, U. S. A. (*Retired*), which, in a report dated August 24, 1920, approved the recommendations of the Division Engineer as to the necessity for remedial works, as follows:

"So far as concerns the preservation of scenic beauty, the most important measure suggested by the Division Engineer is the construction of a weir and certain rock excavation for the uniform distribution of flow over the Horseshoe, * * *. To preserve the flow of the American Fall he recommends a rough submerged weir between the head of Goat Island and the Canadian side, part of which has already been made by dumping dredge spoil.

"We are in accord with these recommendations of the Division Engineer as to the advantage of distributing as nearly uniformly as possible the water flowing over the Horseshoe Fall and as to the general method by which this can be accomplished."

* * * * * *

"No definite limit should be set upon the critical discharge over the Falls and the amount of water permitted to be diverted, other than to state that the remedial works should be designed so as to afford the maximum attainable scenic beauty, in our opinion corresponding to an ultimate diversion of 100 000 to 110 000 cubic feet per second."

Special International Niagara Board.—Finally, in 1926, the Governments of the United States and Canada appointed a Commission, known as the Special International Niagara Board, to study and report on the problem. It is expected that the report of this Board will be issued in 1929 when its recommendations may be added to those already quoted. Pending the preparation of its final report, the Board issued an interim report in December, 1927, which contains interesting recommendations covering the construction of initial remedial works at an estimated cost of $1 750 000, for the preservation of the scenic values of the Falls.

This report led to an agreement between the United States and Canada covering the construction of the remedial works recommended by the Board. The agreement was embodied in a Convention and Protocol signed in Ottawa, Ont., Canada, January 2, 1929, ratified by the Parliament of Canada, May 20, 1929, and, at present (June, 1929), is awaiting ratification by the Senate of the United States.

Scenery and Power.—Evidently, the ultimate development of the Falls of Niagara, for many years at least, will represent a compromise between the demands of scenery and of power. Lest it should be inferred that these are always opposed to one another, it may be noted that the requirements of power actually may be made to serve the needs of scenery. Furthermore, economic pressure surely will demand the same efficiency in the use of water for scenery as for power.

The maximum diversions officially recommended to date, however, are only 100 000 cu. ft. per sec. around the Falls and 40 000 cu. ft. per sec. around the Lower Rapids, provided suitable works are constructed for the preservation of the scenery and the regulation of the river.

Existing Plants.—The distribution of power and use of water in the existing plants is shown on Fig. 1. No attempt has been made to account for small quantities of treaty water, such as those used through the New York State Barge Canal and in the plant of the International Railway in Canada. Neither will the subsequent discussion be complicated by reference to the division of water between the United States and Canada. The principle of equality in respect of the use of boundary waters has already been agreed upon and ultimately the benefits of Niagara diversion will be divided equally between the two countries.

The notes on Fig. 1 show that, with the present allowed diversion of 56 000 cu. ft. per sec., a daily average output of 1 137 000 h.p. and a peak of about 1 388 000 h.p. are being produced. A ratio of 1 : 1.2 between average and peak for the plants on the American side has been fixed by the Federal Power Commission. No ratio has yet been fixed on the Canadian side.

An average of about 16 000 cu. ft. per sec. are being diverted around both the Falls and the Rapids. The remaining 40 000 cu. ft. per sec. are being diverted around the Falls only.

Proposed New Plants.—If new plants using a 312-ft. head are built to utilize the 24 000 cu. ft. per sec. remaining out of the 40 000 now recommended for diversion around the Falls and Rapids, the present plants (1928) are sufficient to utilize the remaining 60 000 cu. ft. per sec. around the Falls. Under these conditions, the ultimate development would give about 2 100 000 h.p. for the daily average and 2 520 000 h.p. on peak. By scrapping the most inefficient plants (marked 2, 3, and 5 on Fig. 1) and building new plants of modern type to utilize the 220-ft. head from above the Upper Rapids to the Maid-of-the-Mist pool, the maximum development under present recommendations would be increased to a daily average of 2 500 000 h.p., and, with the Federal Power Commission's ratio, to about 3 000 000 h.p. on peak. Thus, recommendations have already been made which would permit a total development of power twice as great as that provided by the existing plants.

It is probable, however, that more than 40 000 cu. ft. per sec. may be diverted around the Lower Rapids without serious impairment of their scenic beauty or their ice-discharging capacity. If the total quantity recommended for diversion around the Falls were permitted to be diverted also around the Lower Rapids, the total development would rise to 3 000 000 h.p. average and 3 600 000 h.p. peak. From that point onward the ultimate development of Niagara can be estimated only by determining the minimum quantity of water required to preserve a satisfying scenic spectacle.

When the limits of diversion consistent with the preservation of a satisfactory scenic spectacle are reached, it is probable that it will become necessary to find means to utilize as much as possible of the flow of the river during the night hours when scenic effects may be disregarded. This may be done by installing additional capacity in industrial plants to be supplied with power only during the night hours, or by substituting hydro-electric power for steam power to save fuel that would otherwise be burnt at night, or by storing the water for use at hours of peak load during the day. The greater value of peak power, which results from the instinct that leads most people to prefer to work during the day and sleep at night, may tend to offset the cost of storage. Such expedients as these will probably form part of the ultimate development of Niagara Falls.

It is most important to remember, however, that at this moment it is not at all necessary to decide how far Niagara power development, ultimately, may be carried. The preservation of a satisfactory spectacle may be insured by a step-by-step development of power and scenery together. A diversion of 20 000 cu. ft. per sec. is sufficient water for the first step; this will provide for power development on a sound economic basis. As each development is made, its effect on the scenery may be noted, and if necessary remedial works

may be provided. It is beyond question that the logical step immediately required in the development of Niagara power is the construction of the remedial works recommended by the Board and a new plant on the American side under a head of 312 ft. to utilize initially not less than 10 000 cu. ft. per sec. and produce not less than 300 000 h.p.

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REGULATION OF LEVELS, FLOW, AND NAVIGATION, NIAGARA RIVER:

SUMMARY AND CONCLUSIONS OF VARIOUS STUDIES*

By George B. Pillsbury,† M. Am. Soc. C. E.

Synopsis

The factors relating to the regulation of the Niagara River are summarized and the studies made by various agencies of the regulation of Lake Erie are described. The conclusion is reached that the results of these studies have not indicated the advisability of regulating the flow of the Niagara River.

The purpose of this paper is to discuss the regulation of the Niagara River, with its widespread ramifications, and to relate the efforts that have been made to solve it. Briefly, the problem is this: To find in what manner the natural regimen of the outflow from Lake Erie through the Niagara River can be altered to produce benefits to mankind which will so outweigh the resulting injuries as to warrant the cost of constructing and operating the controlling works.

There is certainly no insuperable engineering difficulty in building a gate structure that will control the outflow of the Niagara. The problem is how to use it if built.

Inflow, Lake Level, and Discharge

The outflow of Lake Erie is now determined by the discharge over the natural rock weir of the rapids at the head of the Niagara River. While this is a submerged weir, the back-water is almost entirely a function of the head, and the discharge varies with the elevation of Lake Erie at Buffalo, N. Y. Within the comparatively narrow limits of range of stage of the lake, the discharge varies substantially as the lake elevation, thus establishing the present balance between the storage in the lake and the outflow. The increas-

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ing discharge gradually checks an undue rise in head from an excessive supply of water to the lake, and the decreasing discharge gradually checks an undue fall when the supply is deficient. At the same time the lake absorbs in storage any temporary fluctuation of supply.

The relation between storage and discharge for Lake Erie may be expressed by the statement that a rise of 1 ft. in the lake will create additional storage equivalent to a flow of 110 000 cu. ft. per sec. for 1 month; and will increase the discharge out of the lake by from 20 000 to 23 000 cu. ft. per sec., depending on the stage.

As a consequence of the present automatic regulation, the extreme variation in the monthly mean levels of Lake Erie, in the sixty-eight years for which continuous and authentic records are available, has been 4.13 ft. The highest monthly mean level was reached in June, 1876; the lowest in December, 1925. During the intervening period the lake levels had been lowered by 0.6 ft., on account of the diversions made by the Chicago Sanitary District, the Welland Canal, and otherwise. Therefore, the fluctuation of the monthly mean elevation of the lake, due to varying supply, has been only 3.5 ft. The corresponding variation in the monthly mean discharge of the Niagara River has swung from a maximum of 252 000 cu. ft. per sec., in June, 1876, to a minimum of 166 000 cu. ft. per sec., in December, 1925.

These lake levels and discharges are monthly means. The water elevation at Buffalo is particularly subject to fluctuation, due to wind and barometric pressure. During a westerly gale the water has been known to rise 8 ft. above its monthly mean level; and, again, it has fallen 4 ft. below its monthly mean. During such periods, discharge of the Niagara River as low as perhaps 110 000 cu. ft. per sec., and as high as more than 300 000 cu. ft. per sec., may obtain.

The variations in the monthly mean levels of the lake are due to fluctuations in supply, that is, to the variations in the sum of the water received from the upper lakes and the run-off of the Lake Erie Basin, less the evaporation. The maximum monthly supply of record to Lake Erie was in April, 1913, corresponding to the rainfall which created the disastrous floods in the tributaries of the Ohio in March of that year. It reached an average of 324 000 cu. ft. per sec. for a month and resulted in a rise of lake level of about 1.6 ft. During August, September, October, and November, the evaporation from Lake Erie usually exceeds the combined rainfall on the lake and the run-off from its own drainage basin, but the supply is well maintained by the inflow from Lake Huron. The minimum supply appears to have occurred in January, 1920, due to the retardation of the inflow by ice obstruction in the St. Clair and Detroit Rivers. It amounted to 126 000 cu. ft. per sec., and was accompanied by a fall of about 0.5 ft. in level.

The total supply to Lake Erie is usually the greatest in April and least in October. The April supply exceeds that of October on the average by about 80 000 cu. ft. per sec. The resulting seasonal fluctuation in the level of the lake averages about 1.5 ft., and the corresponding average seasonal fluctuation in discharge, about 30 000 cu. ft. per sec.

These are the high points of the variations in the discharge of the Niagara, with its present natural regimen, and of the resulting levels of Lake Erie.

Interests Affected by Regulation

Before considering schemes for regulation it is necessary to cast up an account of those who are affected by the discharge of the Niagara River. Since this discharge controls the level of Lake Erie, these include the navigation interests on that lake. Such navigation is a vital part of the general lake traffic. It moves yearly about 45 000 000 tons of iron ore into ports on Lake Erie, or most of that used in the iron industry on the Continent; it moves about 300 000 000 bushels of grain annually; and it moves the coal shipped from Lake Erie to the territory lying north and west of the lakes. Lake commerce is highly organized, and utilizes practically the last inch of depth available. While it is true that the limiting draft, in recent years, has been fixed by the levels of the upper lakes, no system of regulation could be tolerated which proposed to lower either the extreme low, or the average, levels of Lake Erie. On the contrary, studies heretofore made of the regulation of the Niagara have been directed primarily to improving the lake levels.

A second set of citizens of the United States and Canada vitally affected by the levels of Lake Erie is the several million residents of the cities and towns on the shores of the lake, and along the Detroit River. They are interested, primarily, in the high lake levels. The sewer systems of the low-lying districts of the cities are based on the levels of the lake as they exist at present, and would be inadequate if these levels were raised, or even if they were maintained continuously at the highest levels occasionally reached in the past. A large industrial area in Buffalo is subject to overflow at rare intervals by the extreme storm rises of the lake. If the lake levels were substantially raised, the losses would become more frequent and more extensive. It is certain that no substantial increase in the high levels of the lake, either in height or in frequency, would be tolerated.

The third class very directly concerned in the regulation of the Niagara is the vast number of those who come to enjoy the scenic beauty of the Falls, as well as those, on both sides of the border, concerned in the development of power. Their interest in an unimpaired discharge of the Niagara is too obvious to require elaboration. It is probably more important to them that what may be termed the ordinary flow be not reduced, than that occasional extreme low flows be improved.

A fourth group are those on Lake Ontario and on the St. Lawrence. Probably the most important are the navigation interests in the Port of Montreal, Que., Canada. The low-water elevation in that port is 18 ft. or more, above sea level, and depends on the slope of the St. Lawrence below Montreal, which, in turn, is maintained by the volume of discharge. While it is true that the storage in Lake Ontario will "damp out" any temporary fluctuations in the outflow of the Niagara, yet any systematic retardation, continuing over several months, will be felt at Montreal. It is not enough that the minimum flow be kept at a point higher than that to which it has actually fallen in the past; the normal average low-water flow during August, September, October, and November must be maintained. If the normal flow of the Upper St. Lawrence

is reduced, an abnormal low discharge of the Ottawa River may result in record low levels in the harbor.

An increased discharge of the Niagara in winter may affect the flood heights during the spring break-up at and below Montreal. Large sums have been expended to guard against the consequences of these floods.

The future development of power, in a comprehensive way, on the St. Lawrence above Montreal will entail the control of the outflow of Lake Ontario, through the turbines of power plants and the sluice-gates of dams. It is clear that this control would be affected, perhaps favorably and perhaps unfavorably, by variations in the inflow into Lake Ontario through the regulation of the Niagara.

In short, many thousands of citizens of the United States and Canada, extending from Detroit, Mich., to Quebec, Que., have based their industry and their mode of existence, to varying degree, on the regimen of the Niagara as now established by Nature. The effect of any change must be considered before a scheme for regulation can be accepted by the two Governments concerned. If the direct benefits prove large and certain, a way will eventually be found to overcome the resistance of those who will be, or fancy they may be, injured to some degree directly or indirectly. If, on the other hand, the direct benefits are small, and realizable by other means at a cost commensurate with the cost of regulation, then it is evident that widespread indirect disadvantages will throw the balance against regulation.

An Error of Viewpoint

A misconception exists as to the extent of the improvement in lake levels that might be accomplished by regulating works. The diversion of the Chicago Sanitary District may be placed at 8 500 cu. ft. per sec., that through the Welland Canal (Niagara River) at 3 000 cu. ft. per sec., and the equivalent effective diversion through the Black Rock Canal in the Niagara at 1 000 cu. ft. per sec., a total of 12 500 cu. ft. per sec. The result of these diversions is that the levels of Lake Erie are steadily 0.6 ft. lower than they would be had the diversions not been made. By constructing fixed works which will so contract the Upper Niagara River as to reduce its discharge capacity by 12 500 cu. ft. per sec. at all stages, the loss of 0.6 ft. can be restored without otherwise controlling the discharge. In other words, the low level of the lake, occurring subsequent to the diversions, would be raised by 0.6 ft. without increasing the high levels, which occurred prior to the diversions.

When, therefore, a system of regulating works is studied, and is found with the supplies of record, to raise the low levels of the lake by, say, 1.6 ft. without raising the high levels of record, it is evident that the 0.6 of the amount should not be credited to the regulation as such, but is the equivalent of a constant retardation which could equally well be secured by fixed works, at far less cost. The benefit attributable in such case to the installation of manually controlled gates is 1.0 ft. and not 1.6 ft.

Official Studies

Apparently the first serious proposal for the regulation of Lake Erie was made in 1900 by the Board on Deep Waterways, which had been specially

appointed to prepare plans and estimates for securing deep-draft navigation from the Great Lakes to New York Harbor. This Board proposed gates at the head of the Niagara River, which would hold Lake Erie at substantially the level of 574.7. This elevation is slightly higher than the maximum monthly mean level since 1860. It is only once in twenty years that the lake reaches within a foot of this elevation under the natural regimen of the Niagara River. It is very clear that so high a level would not now be tolerated.

Lake Erie can be held at a fixed level solely by discharging water from it as fast, and only as fast, as it is received. The discharge capacity of the Niagara, neglecting the retardation due to any gate piers, is 257 600 cu. ft. per sec. when Lake Erie is at Elevation 574.7. The supply for March and April, 1913 (296 000 and 324 000 cu. ft. per sec., respectively), would have raised the lake by nearly a foot with all regulating gates wide open. It is not possible to discharge water out of Lake Erie as fast as it may be received, unless the discharge capacity of the Niagara is greatly enlarged. In any event the lake can be held at substantially a fixed level only by greatly increasing the fluctuations of the discharge of the Niagara by an average of 50 000 cu. ft. per sec. each year and substantially reducing the ordinary low-water flow.

The question of the regulation of Lake Erie and of the Great Lakes, in general, was reviewed by the International Waterways Commission, a joint board of American and Canadian engineers. Its report, made in 1910, is a monumental study of the regimen of the Great Lakes. It pointed out the impracticability of regulating the outflow of Lake Erie so as to maintain a practically constant elevation of the lake, but presented a study of regulation based on the supplies of record, which was designed to hold Lake Erie within a range of 2.5 ft., as compared with the natural range, up to that time, of 3.8 ft. With the system proposed, there would have been an improvement of about 1 ft. in the ordinary low-water navigable depths on Lake Erie, about 0.3 ft. of which would have been due, however, to correcting the effects of diversions then existing. This improvement in the levels of Lake Erie would be at the cost of throttling the flow of the Niagara to its record minimum for a considerable part of the time, of increasing the fluctuations in Lake Ontario and in the St. Lawrence, and of lowering the minimum recorded level on Lake Ontario by about 4 in. In view of these counterbalancing injuries, the recommendation of the International Waterways Commission was adverse to the construction of regulating works in the Niagara River, or elsewhere.

Other Proposals

Following this report various unofficial studies were made of the regulation of Lake Erie, some of which indicated that, with a suitable manipulation of the outflow, the disadvantages of the system studied by the International Waterways Commission could be overcome. The increasing demand for power, and the large amounts of power derivable from the head in the Niagara and St. Lawrence Rivers, pointed to the need for studying the manipulation of the levels of the lakes to develop the maximum useful storage, and the greatest steady outflow, in order to increase the potential output of primary power.

In 1919, Francis C. Shenehon, M. Am. Soc. C. E., made a report to the Chicago Sanitary District setting forth a scheme of regulation designed to equalize the summer flow of the Niagara River. As has been pointed out, the natural regimen affords an outflow which is ordinarily greatest in June and least in December. By retarding the flow after the danger of high water is past, it is possible to increase the autumn discharge. The results of the system studied indicated a gross gain of 19½ in. in the low levels of Lake Erie and of 12 in. in the high levels. The regulation, however, was based on an assumed diversion of 10 000 cu. ft. per sec. by the Chicago Sanitary District, and this diversion, with that of the Welland Canal and others, from Lake Erie would lower the lake by about 9 in. The effect of the proposed system of regulation would be, therefore, to raise the low levels by 10 in., and the mean levels by 3 in. more than could be effected by relatively inexpensive fixed works. The effect on the outflow of the Niagara is shown by the duration curves in Fig. 1.

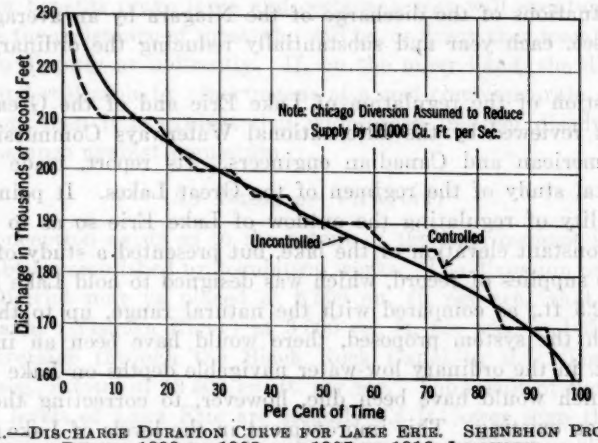

FIG. 1.—DISCHARGE DURATION CURVE FOR LAKE ERIE. SHENEHON PROPOSAL. PERIOD, 1893 TO 1902 AND 1907 TO 1916, INCLUSIVE.

In a paper* on the St. Lawrence Waterway before the Society, in 1924, Mr. Shenehon pointed out that the supply of water to the Great Lakes was sufficient to produce a steady outflow of 230 000 cu. ft. per sec., with a moderate annual hold-over storage. This is about 36 000 cu. ft. per sec. in excess of the present unrestricted minimum outflow.

In 1925 an Engineering Board of Review engaged by the Chicago Sanitary District prepared a scheme for the regulation of all the lakes, including Lake Erie. A careful review of the published computations and diagrams accompanying the report, and contained in a supplementary report of John R. Freeman, Past-President, Am. Soc. C. E., leads to the conclusion that the results of the system of regulation there considered would not differ substantially from those shown by the tentative study made by the Joint Board of Engineers on the St. Lawrence Waterway (presently to be discussed) so far

* *Transactions*, Am. Soc. C. E., Vol. 89 (1926), p. 444.

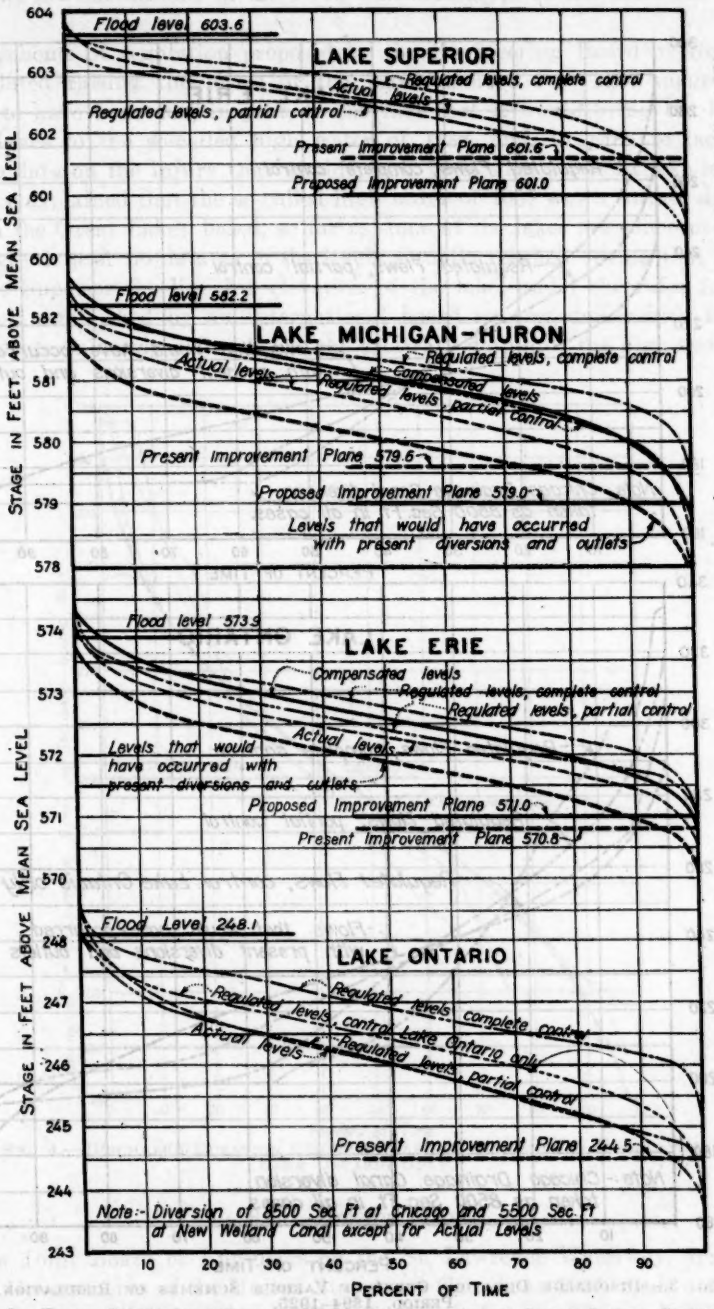

FIG. 2.—STAGE DURATION CURVE OF VARIOUS SCHEMES OF GREAT LAKES REGULATION, 1894–1925. NAVIGATION SEASON, APRIL 1 TO DECEMBER 1.

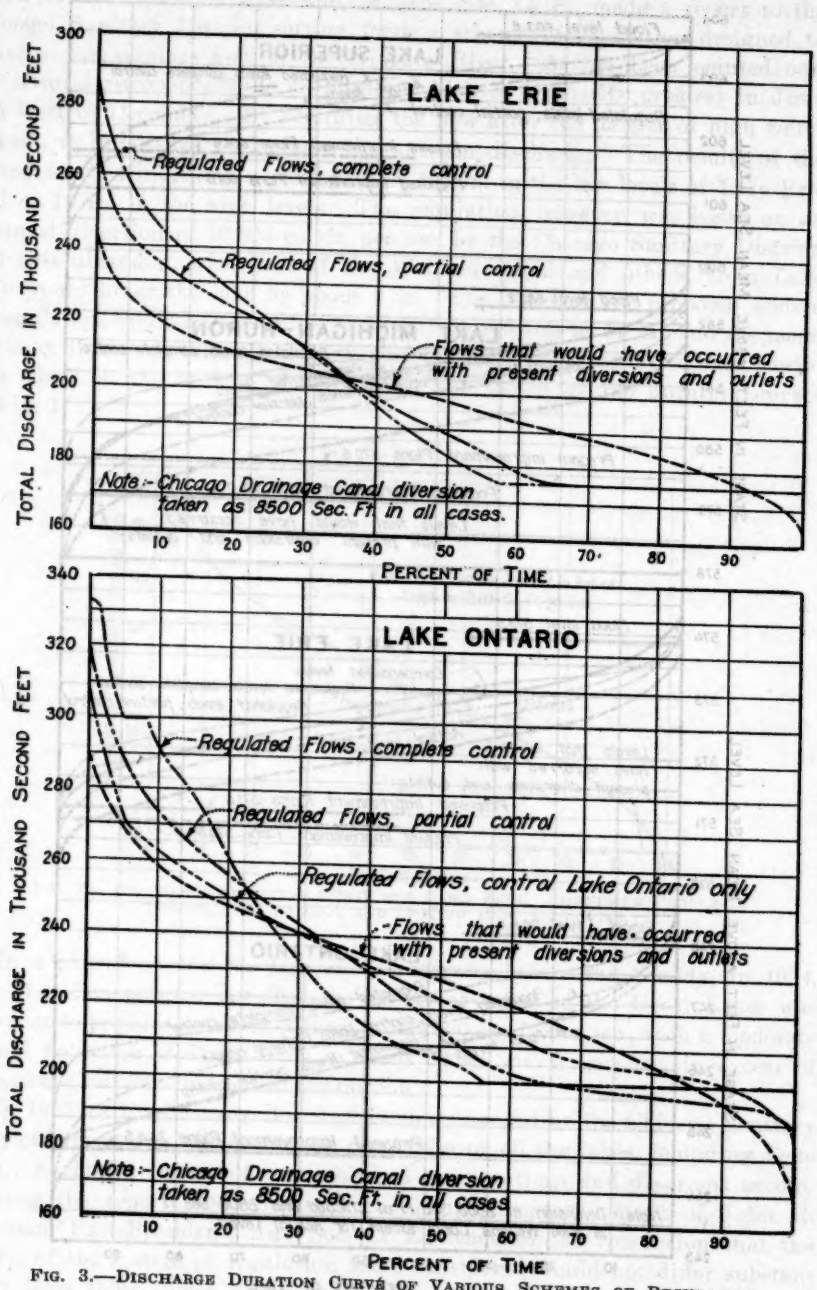
LAKE ERIE
Regulated Flows, complete control
Regulated Flows, partial control
Flows that would have occurred with present diversions and outlets
Note:- Chicago Drainage Canal diversion taken as 8500 Sec. Ft. in all cases.
TOTAL DISCHARGE IN THOUSAND SECOND FEET
PERCENT OF TIME
LAKE ONTARIO
Regulated Flows, complete control
Regulated Flows, partial control
Regulated Flows, control Lake Ontario only
Flows that would have occurred with present diversions and outlets
Note:- Chicago Drainage Canal diversion taken as 8500 Sec. Ft. in all cases
TOTAL DISCHARGE IN THOUSAND SECOND FEET
PERCENT OF TIME

FIG. 3.—DISCHARGE DURATION CURVE OF VARIOUS SCHEMES OF REGULATION. PERIOD, 1894–1925.

at least as the fluctuations in the levels and discharges of the lakes are concerned.

The scheme of regulation proposed by the Engineering Board of Review contemplated raising the levels of the lakes about 2 ft. This suggestion appears to have been due, in part, to the fact that it would bring the high-water planes to the so-called high water of 1838, and, in part, to lack of available data on the injury that would result from such a rise in lake levels. It may be explained that the so-called high water of 1838 was a former datum plane on the Great Lakes, based, so far as some of the lakes are concerned, on incomplete data or no data as to the levels actually reached during that year. On Lake Superior the limiting elevation of the lake, under the rules for its regulation established by an international board in accordance with treaty requirements, is nearly a foot below the arbitrary datum of the high water of 1838.

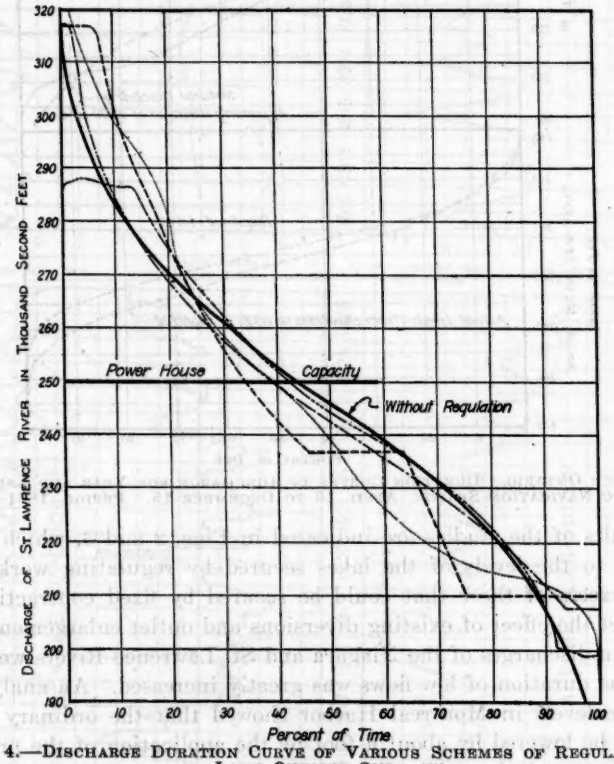

FIG. 4.—DISCHARGE DURATION CURVE OF VARIOUS SCHEMES OF REGULATION, LAKE ONTARIO ONLY.

ST. LAWRENCE BOARD

The Joint Board of Engineers on the St. Lawrence Waterway, appointed by the Governments of the United States and Canada in 1924, made a detailed study of the possibilities of regulating the Great Lakes. This included a full investigation into the permissible high levels of the lakes, the maximum and

minimum discharges obtainable or permissible in the interconnecting rivers, and a revision of existing data on the water supply of the lakes since 1860, in the light of the latest evidence. With these data a tentative scheme of regulation, based on the budgeting of water between the lakes, was presented in detail. A modified program, based on a less comprehensive control of the outflow of Lake Huron, was also evolved.

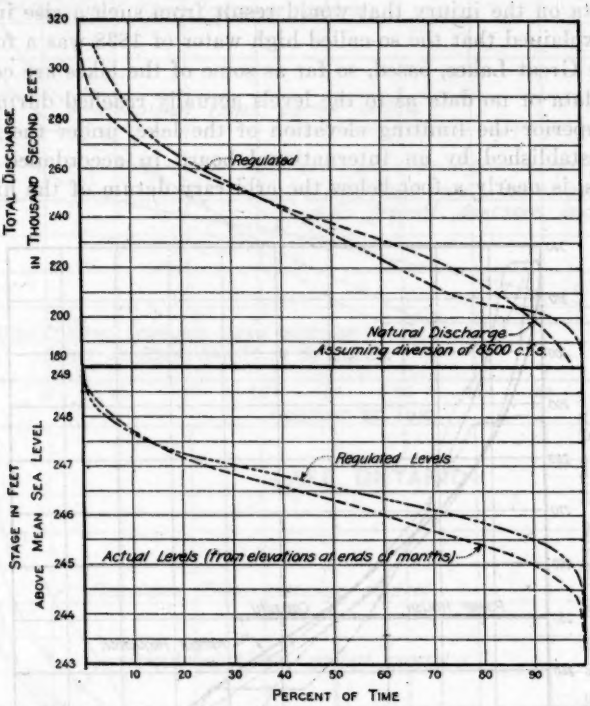

FIG. 5.—LAKE ONTARIO. DURATION CURVES OF DISCHARGE FOR YEAR AND LAKE LEVELS DURING NAVIGATION SEASON, APRIL 15 TO DECEMBER 15. PERIOD, 1861–1925.

The results of the studies are indicated in Figs. 2 and 3, which show that the benefits to the levels of the lakes secured by regulating works, are not greatly in excess of those that could be secured by fixed contractions which would correct the effect of existing diversions and outlet enlargements. While the minimum discharges of the Niagara and St. Lawrence Rivers were slightly increased, the duration of low flows was greatly increased. An analysis of the effect on the levels in Montreal Harbor showed that the ordinary low-water levels would be lowered by about a foot by the application of the program for comprehensive regulation. The benefits from regulation, when fully analyzed, were obviously such as to leave no ground for a recommendation that regulating works be constructed.

CONCLUSIONS

The regulation of Lake Erie may be directed to reducing the fluctuation in its levels, or to altering the natural fluctuation of its outflow. It appears

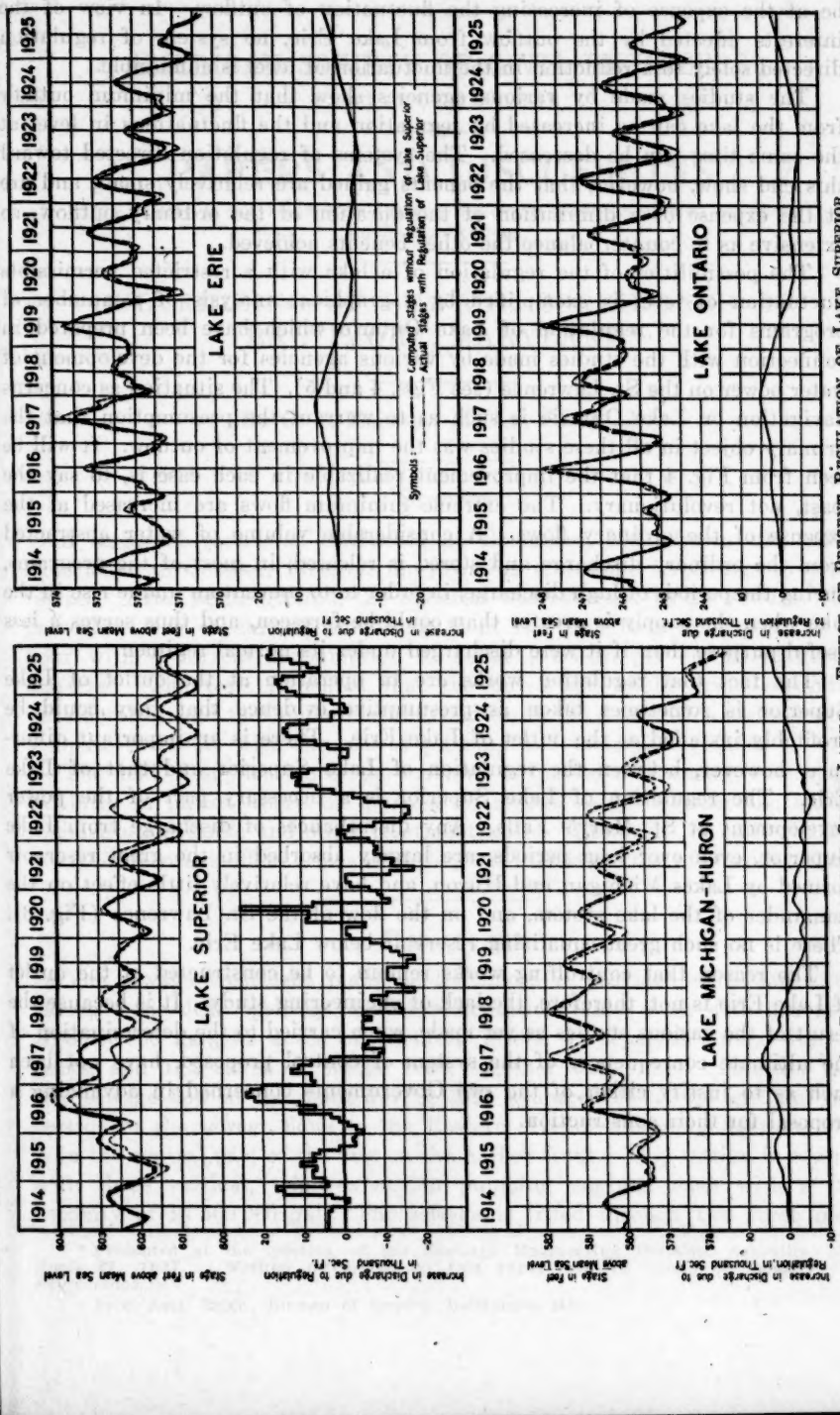
LAKE SUPERIOR
LAKE MICHIGAN-HURON
LAKE ERIE
LAKE ONTARIO
Stage in Feet above Mean Sea Level
Increase in Discharge due to Regulation in Thousand Sec. Ft.
Symbols: Computed stages without Regulation of Lake Superior
Actual stages with Regulation of Lake Superior
1914 1915 1916 1917 1918 1919 1920 1921 1922 1923 1924 1925

FIG. 6.—REGULATION OF THE GREAT LAKES. EFFECT OF PRESENT REGULATION OF LAKE SUPERIOR.

fundamental that a reduction in the fluctuation of the levels of a lake must be at the expense of increasing the fluctuation of outflow. In view of the interests affected by the outflow from Lake Erie, no system of regulation directed solely to a reduction in the fluctuation of level is admissible.

The studies made by various agencies show that the minimum outflow from the lake can be increased by regulation and the fluctuations in level at the same time can be decreased. The systems of regulation directed toward this end show, however, that the benefits gained are relatively small, and are at the expense of a diminution of the duration of the ordinary outflow, so extensive as to counterbalance the other benefits achieved.

The possibilities of the regulation of a lake with a restricted permissible fluctuation of level is exemplified by a graphical analysis of a number of programs for the regulation of Lake Ontario which have been prepared in connection with the studies made by various agencies for the development of water power on the St. Lawrence (see Figs. 4 and 5). The situation as concerns navigation on Lake Ontario is such as to warrant the presumption that the primary object in all these studies was the improvement of outflow. It will be seen from Fig. 4 that the improvement realizable in each case is, to say the least, not revolutionary. The extreme minimum flows are increased at the expense of the ordinary flows. A considerable volume of water abstracted from the ordinary discharge and stored is released, in most of the programs, during the periods of high discharge, in order to to prevent an undue rise in the lake when the supply is greater than could be foreseen, and thus serves a less useful purpose than if it were discharged under its normal regimen.

The fact that regulation works are in operation at the outlet of Lake Superior is sometimes taken as presumptive evidence that they could be profitably installed at the outlet of Lake Erie. There is an important difference, however, between the regulation of Lake Superior and that of Lake Erie. The regulation of Lake Superior is a necessary part of the power development at St. Mary's Falls. Any disturbances of discharge from Lake Superior, even over long periods, are largely absorbed in the great reservoir formed by Lakes Michigan and Huron, and have relatively little effect on the remainder of the lake system, and on the flow of the St. Lawrence (Fig. 6). There is no such great equalizing reservoir below Lake Erie.

The reason that controlling works remain to be constructed at the outlet of Lake Erie is not, therefore, the lack of engineering study. It is because the result of the various studies as yet made, when carried to the determination of the ultimate consequences of the system of control proposed, have not been such as to justify either of the two Governments concerned in advancing a proposal for their construction.

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PULVERIZING OF SEWAGE SCREENINGS AT BALTIMORE, MARYLAND*

By C. E. Keefer,† M. Am. Soc. C. E.

Synopsis

A series of tests was conducted at Baltimore, Md., during the period from June to October, 1926, on the grinding of sewage screenings in a pulverizer. After being ground, the material was returned to the sewage. The pulverizer was capable of shredding from 1.5 to 2 tons of screenings per hour with a current consumption of about ¼ kw-hr. per cu. ft. of ground material. The tests indicated that the pulverizer would grind screenings satisfactorily, and that under certain conditions, the cost of this method would compare favorably with the cost of pressing and incinerating them.

The field of application of the machine is limited to a certain extent, particularly in places where the purpose of screens is to decrease the load on sedimentation tanks and other treatment units. However, the pulverizing of sewage screenings offers a means of disposal that is worthy of consideration.

Introduction

The sewage treatment plant at Baltimore is provided with bar screens having ⅞-in. clear openings, four sedimentation tanks, each with a capacity of 3 800 000 gal., 30 acres of trickling filters, 51 separate sludge digestion tanks, and 11.3 acres of sludge beds, together with other treatment units. The sewage flow at the plant during 1925 averaged 55 090 000 gal. daily. A portion of the sewage flows to the Eastern Avenue Pumping Station, which is in the central part of the city on the harbor front. This station is equipped with three vertical, triple-expansion pumping engines, each with a daily capacity of 27 500 000 gal. The sewage is lifted through two force mains,

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and becomes a part of the flow which discharges by gravity at the sewage works. During 1925, the flow at the pumping station averaged 24 690 000 gal. per day. Before the sewage reaches the pumps, it passes through four cage screens, which have clear openings of 1 in. between the bars.

The screenings at the pumping station are pressed and then incinerated, while at the sewage works they are sold to farmers for 25 cents per load. This latter method of disposal is not entirely satisfactory because the screenings, which accumulate for several hours before they are removed by local farmers, tend to produce odors and attract flies. Furthermore, there is a certain health hazard in spreading this material over the ground for fertilizing purposes. There is also some question as to the efficacy of the screenings when used as a fertilizer.

Since a new screening plant, which will consist of three automatically cleaned coarse screens, each with a maximum capacity of 150 000 000 gal. daily, is being planned for the sewage works, a study was made to determine the most satisfactory method of disposing of the screenings. It was decided to conduct a series of experiments on grinding the screenings by means of a pulverizer. This was suggested by the method formerly practiced at Berlin, Germany, of crushing the screenings into a pulp and returning them to the sewage.

Historical

So far as the writer knows, the method of pulverizing screenings and returning them to the sewage has never been tried except at the sewage pumping station of Radial System No. 11, in Berlin. There the screenings were passed through a series of crushing rolls and, after being macerated, were returned to the sewage on the up-stream side of the screens.

Scope of Baltimore Experiments

A 24 by 18-in. pulverizer (see Fig. 1) was installed, first at the treatment works, and, later, at the pumping station. A series of experiments was conducted to determine: (1) Whether the machine would pulverize sewage screenings satisfactorily; (2) whether the ground material would have any detrimental effect on the operation of the sedimentation tanks, which receive the screened sewage at the treatment works, or on the efficiency of the pumps at the pumping station; (3) whether the pulverizer was of a sufficiently sturdy design to withstand hard use; and (4) whether the operation of the machine would be economically feasible.

Description of Pulverizer

The pulverizer has a rated capacity of from 1 to 2 tons per hour. As indicated by Figs. 1 and 2, it is box-shaped, with an opening in the top, into which the unground material enters. The machine is approximately 5 ft. by 2 ft. 6 in. in plan, and stands 2 ft. 10 in. high. The outer housing consists of cast-iron plates, which have machined joints bolted together. The front of the machine is provided with a hinged steel cover, held in place by C-clamps. When this cover is raised, the inside of the machine can be examined. The interior

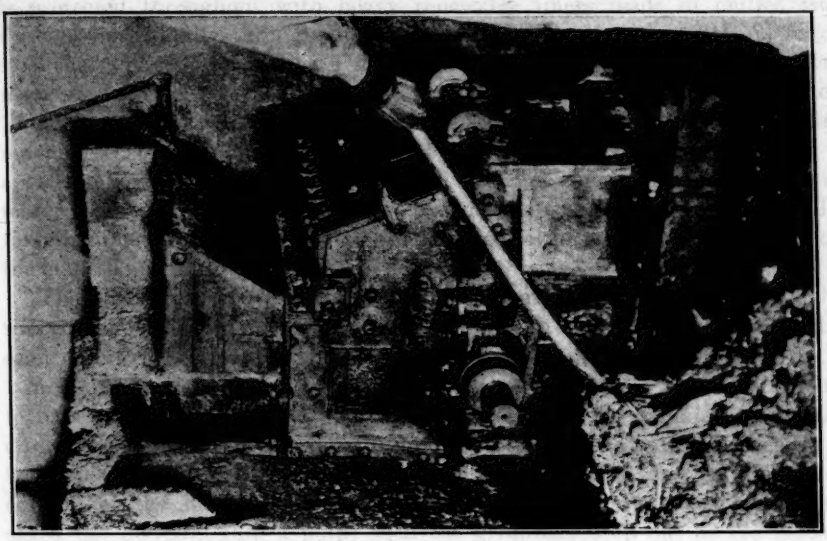


FIG. 2.—VIEW OF PULVERIZER IN ACTION.

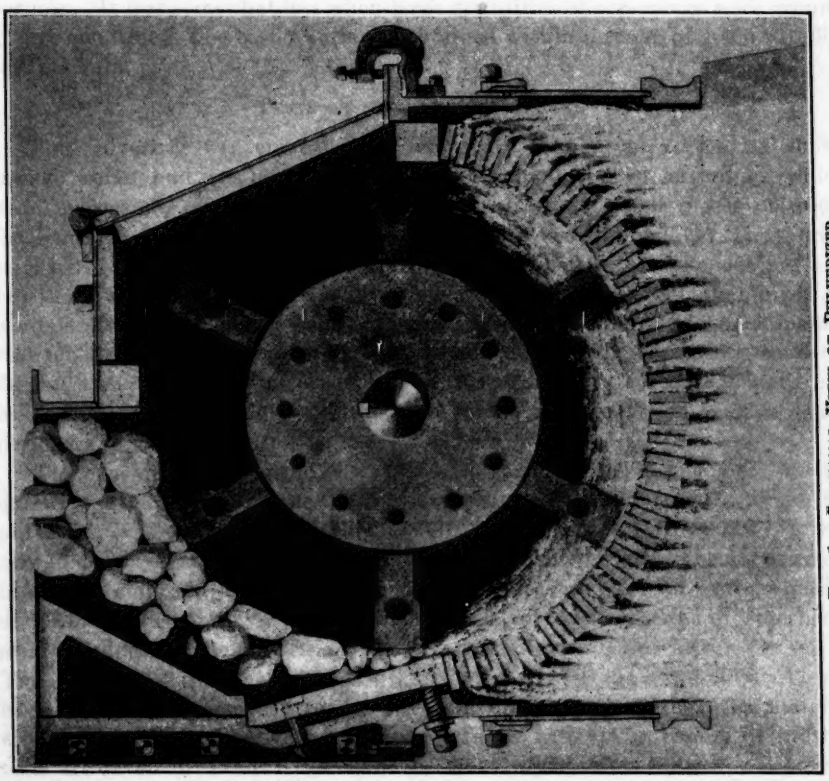
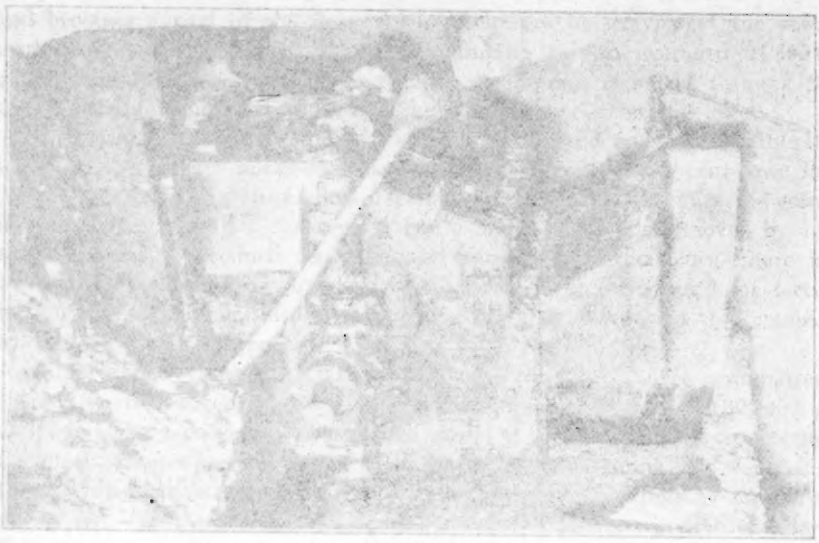
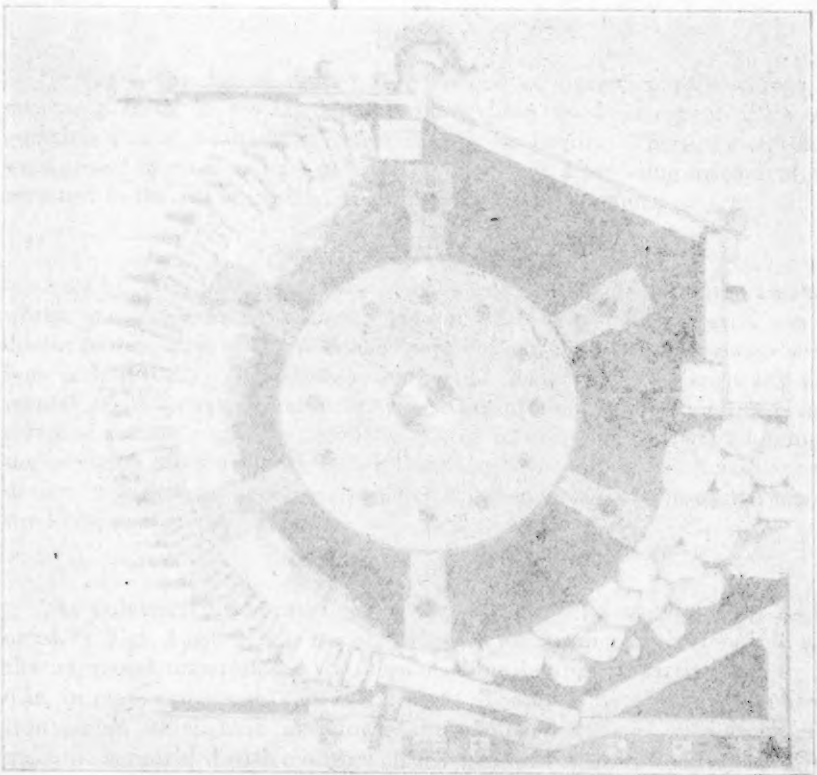


FIG. 1.—SECTIONAL VIEW OF PULVERIZER.





is protected throughout with heavy renewable liners made of chilled iron. Passing horizontally through the center of the pulverizer is a 2$\frac{15}{16}$-in., forged steel shaft. As shown in Fig. 1 this shaft passes through the center of a rotor made of cast-steel end disks and nine plate-steel center disks, which are spaced 1$\frac{5}{8}$ in. apart. Between each pair of plates there are four sets of free-swinging, manganese steel hammers. One end of each hammer is secured to a disk by a steel pin, while the other end swings free and comes in contact with the material to be ground. In the course of time the free end becomes worn. The forty hammers (8$\frac{3}{4}$ by 2$\frac{1}{2}$ by $\frac{5}{8}$ in.) are designed so that they can be reversed end for end. Thus, their useful life is doubled.

The lower part of the interior of the pulverizer is fitted with a circular-shaped cage made of rectangular steel bars, with sharp edges set parallel to each other. The clear spacing between the bars can be varied from $\frac{1}{8}$ in. to 1 in. This bar cage serves to retain the material until it is ground sufficiently to pass out of the machine. As the rotor revolves, the hammers swing in a circle perpendicular to the longitudinal direction of the bars. The clearance between the ends of the hammers and the bars is from $\frac{1}{4}$ to $\frac{1}{2}$ in. The blunt corners of the hammers in conjunction with the sharp edges of the bars shred the material into a pulp; then the grindings drop out of the machine between the bars. During the tests the speed was approximately 1 450 rev. per min. It was somewhat less when large quantities of screenings were ground at any one time. The best results are obtained within a range of 1 400 to 1 600 rev. per min.

Tests at the Sewage Works

The pulverizer was erected on a temporary foundation at the Baltimore sewage works, and was belt-driven by a 30-h.p. motor. The screenings, which were macerated, consisted of rags, paper, fruit skins, small pieces of wood, fecal matter, etc. The test may be divided into two groups: (1) Those with steel bars separated by 1-in. clear openings in the bottom of the machine; and (2) those with bars separated by $\frac{3}{4}$-in. openings. In Group (1), the pulverizer was operated 6 days, and 35 450 lb. of screenings were ground. The machine was run for about 1$\frac{3}{4}$ hours each day, and in this time all the screenings collected during that day were pulverized. For this group, the screenings were collected and the sewage flow was measured for an average period of 22.83 hours per day.

The capacity of the machine was 3 335 lb., or 62.71 cu. ft. per hour, and the quantity of current used was 0.255 kw-hr. per cu. ft. of screenings. When the pulverizer was equipped with bars having $\frac{3}{4}$-in. openings (Group (2)), it was operated 7 days, and 38 752 lb. of screenings were pulverized. Then the screenings were collected and the sewage flow was measured for an average period of 23.43 hours per day. Moreover, 3 454 lb., or 64.95 cu. ft., of screenings were shredded per hour with a current consumption of 0.254 kw-hr. per cu. ft. The capacity of the machine was slightly greater with the more closely spaced bars. The only apparent reason for this difference was that the material was put into the pulverizer at a slightly faster rate. The results of the two tests are given in Table 1.

TABLE 1.—Quantity of Screenings Collected and Shredded, and Current Used by Pulverizer.

Date 1926.	Sewage flow, in million gallons.	Screenings Collected and Pulverized.							Current Used.		
		Volume.		Weight, in pounds.	Time required to pulverize.		Cubic feet per hour.	Pounds per hour.	Total quantity, in kilowatt-hours.	Kilowatt-hours per cubic foot.	Kilowatt-hours per 1 000 lb.
		Cubic feet.	Cubic feet per million gallons.		Hours.	Minutes.					
(1)	(2)	(3)	(4)	(5)	(6)		(7)	(8)	(9)	(10)	(11)
Group (1).—Bars Spaced 1 Inch Apart.											
June 21...	53.24	99.20	1.87	5 275	1	30	66.14	3 517	30	0.302	5.69
June 23...	53.68	124.30	2.32	6 610	1	23	90.07	4 790	23	0.185	3.48
June 25...	55.63	125.94	2.26	6 697	1	45	71.97	3 827	34	0.270	5.08
June 28...	46.23	108.47	2.34	5 768	2	10	49.98	2 658	30	0.277	5.20
June 30...	58.73	111.27	1.90	5 917	1	38	68.26	3 630	28	0.252	4.73
July 2...	53.01	97.47	1.84	5 183	2	12	44.30	2 356	25	0.256	4.82
Average.	.....	.....	2 09	.....	..	....	62.71	3 335	..	0.255	4.80
Group (2).—Bars Spaced ¾ Inch Apart.											
July 7....	60.63	100.15	1.65	5 326	1	25	70.53	3 751	26	0.260	4.88
July 9....	56.60	81.09	1.44	4 312	1	18	62.38	3 317	27	0.333	6.26
July 12....	60.41	89.55	1.48	4 762	1	47	50.30	2 675	23	0.257	4.83
July 13....	63.70	122.23	1.92	6 500	1	53	65.01	3 457	29	0.237	4.46
July 14....	62.76	124.41	1.98	6 616	1	44	71.91	3 824	31	0.249	4.69
July 15....	56.47	116.74	2.07	6 208	1	41	39.49	3 695	27	0.231	4.35
July 16....	64.12	94.55	1.47	5 028	1	25	66.58	3 541	22	0.233	4.37
Average.	.....	.....	1.72	.....	..	...	64.95	3 454	..	0.254	4.77

In order to grind the screenings it was necessary to introduce a small flow of water into the hopper above the machine; otherwise, they collected in the pulverizer and would not drop out. The composition of the screenings varied considerably. Most of the material was characteristic of that removed from coarse bars, but some of it consisted of masses of rags 3 or 4 ft. long. It was necessary to feed the machine at a uniform rate, but rags had to be ground more slowly in order not to stall the pulverizer.

The average solid content of nine samples of ground screenings was 9.3 per cent. The quantity of water in the material depended largely on the quantity introduced into the pulverizer. The ground material had the appearance of fresh sludge. Some of it was dried, and the maximum length of the stringy material which resulted from the grinding of rags, was determined. When the ¾-in. screen bars were used, the maximum length was 5 in., and with the 1-in. screen bars, the length was 12 in. A large portion of the material, however, was ground into a fine pulp. Two possible ways of disposing of the pulverized screenings are available at the sewage works. It can either be dumped again into the raw sewage and allowed to settle out in the sedi-

mentation tanks, or added directly to the sludge in one of the separate digestion tanks. The latter method most likely will be adopted at Baltimore.

Estimated Cost of Pulverizing Screenings.—The most important question in a comprehensive study dealing with the advisability of pulverizing screenings is a comparison of the total cost of such a process with other established processes. Pressing and incinerating the screenings seems to be the most satisfactory alternative method of disposal at the Baltimore sewage works. The following data have been used as a basis in preparing the cost estimates:

Average daily sewage flow, in gallons	55 000 000
Contributing population	700 000
Quantity of screenings collected, in cubic feet per million gallons	1.87
Weight of screenings, in pounds per cubic foot	53.18
Percentage of dry solids in unground screenings	19.1
Sludge digestion space, in cubic feet	2 800 000
Drying-bed area, in square feet	525 000

As stated previously, the pulverized screenings would be added to the sludge in the separate digestion tanks. If this plan were adopted, additional sludge digestion and drying bed units should be provided. To determine the additional sludge facilities needed, the screenings were weighed and measured in a box, and the sewage flow was observed at the same time by Venturi meters. The quantity of screenings averaged 1.87 cu. ft. per 1 000 000 gal. of sewage, and weighed 53.18 lb. per cu. ft. The dry solids in the screenings as determined from fourteen samples were 19.1%, by weight. There are, therefore, 19.0 lb. of dry solids as screenings removed from each 1 000 000 gal. of sewage. The sedimentation tanks removed 184 parts per million, or 1 535 lb., of dry solids per 1 000 000 gal. of sewage from 1921 to 1924, inclusive. The weight of screenings, therefore, is 1.24% of the weight of the sludge removed by the sedimentation tanks when both materials are reduced to the same moisture. It is estimated that 2 800 000 cu. ft. of sludge digestion space and 525 000 sq. ft. of sludge drying beds are necessary at Baltimore. When these quantities are increased by 1.24%, the added treatment facilities required are 34 720 cu. ft. of digestion space and 6 500 sq. ft. of sludge drying area. This calculation is based on the assumption that the specific gravity of the ground screenings and the sludge is the same.

The following is the estimated cost of these improvements, together with the cost of the pulverizing equipment and the space allotted in the screen building to house it:

Building: 3 200 cu. ft., at 25 cents	$800	
One Type "A" pulverizer	600	
One 30-h.p. motor and auxiliary equipment	610	
Installing equipment, etc.	190	
Total		$2 200
34 720 cu. ft. of digestion space, at 20 cents		6 944
6 500 sq. ft. of sludge beds, at 50 cents		3 250
Total		$12 394

The annual interest charge on these items, at 4½%, is $557.73, and the depreciation is:

Building, $800, at 1⅓%	$10.67
Motor and pulverizer, $1 400, at 9%	126.00
Sludge tanks, $6 944, at 2%	138.88
Sludge beds, $3 250, at 4%	130.00
Total	$405.55

In addition to the grinding, the screenings must be removed from digestion tanks to drying beds, and from drying beds to a sludge dump. Here, the material is loaded on farmers' trucks by a locomotive crane. The quantity of this material is based on the assumption that the volume of sludge removed by the sedimentation tanks is increased by 1.24 per cent. Since the volume of raw sludge from 1921 to 1925, inclusive, on a 90% moisture basis, has averaged 9.09 cu. yd. per 1 000 000 gal. of sewage, the quantity of screenings added to the sludge would be 2 260 cu. yd. each year. When reduced to 68% moisture (which is the average in the sludge removed from the drying beds), and also when a reduction of 10% in volume is assumed for the digestion of the organic material in the screenings, the quantity removed from the sludge beds per year becomes 636 cu. ft. The annual cost of handling this material is given in the following estimate:

Fixed Cost:		
Interest on investment, $12 394, at 4½%	$557.73	
Depreciation and maintenance	405.55	
Total fixed cost		$963.28
Operating Cost:		
Electric current, 9 573 kw-hr., at 2.8 cents	$268.04	
Removing sludge from drying beds, 636 cu. yd., at 55 cents	349.80	
Removing sludge from storage pile, 636 cu. yd., at 25 cents	159.00	
Total operating cost		$776.84
Total cost of handling		$1 740.12

The yearly quantity of electricity used is based on the assumption that 37 540 cu. ft. of screenings are pulverized with a current consumption of 0.255 kw-hr. per cu. ft.

Estimated Cost of Pressing and Incinerating Screenings.—An estimate of the cost of pressing and incinerating the screenings has been computed. The data used in preparing this estimate have been based on experience obtained at Baltimore in pressing and incinerating screenings at the Eastern Avenue Sewage Pumping Station. The following is the estimated cost of a screening press, an incinerator, and the additional cost of a larger building to contain these units:

Building, 11 800 cu. ft., at 30 cents	$3 540
Sewage screening press	3 600
Incinerator, chimney, etc	1 800
Total	$8 940

The interest on the investment at 4½% per annum is $402.30, and the depreciation at 5½% is $491.70.

A record was kept at the pumping station of the quantity of coal used to incinerate the screenings, and the ratio between the volume of unpressed and pressed material. As shown in Table 2, 11.07 lb. of bituminous coal were required to incinerate 1 cu. ft. of screenings, and the volume of material was reduced 47.85% by pressing. With a sewage flow of 55 000 000 gal. per day and 1.87 cu. ft. of screenings removed per 1 000 000 gal., the quantity of coal used yearly would be 97 tons. The following is the estimated total cost of pressing and incinerating the screenings:

Fixed Cost:		
Interest on investment, $8 940, at 4½%	$402	
Depreciation and maintenance, $8 940, at 5½%	492	
Total fixed cost		$894
Operating Cost:		
97 tons of coal at $6.45 per ton	$626	
Power for pressing screenings	10	
Removing ashes	20	
Total operating cost		$656
Total		$1 550

When a comparison is made between the estimated cost of grinding the screenings and pressing and incinerating them, the latter method of disposal is cheaper by approximately $190 per year.

In these estimates no allowance was made for the labor charge involved. In either instance this work would be done by the attendant who cleans the screens, and no extra laborers would be employed. Less work is actually involved in grinding the screenings than in pressing and incinerating them. It is understood, however, that later they have to be handled in the form of sludge.

Tests at the Eastern Avenue Sewage Pumping Station, Baltimore, Md.

After the experiments at the sewage works were completed, the pulverizer was erected at the pumping station and tested from September 14 to October 7, 1926, inclusive. Although careful records were kept of the capacity and the power consumption of the machine, the point of particular interest was to observe whether the ground screenings affected, in any way, the operation of the pumps in the station.

Two of the four cage screens were lifted every half hour during the day. The screenings were removed, weighed, and the volume measured in a box. Samples were collected, and analyzed to determine the solid content. The screenings were then put into the pulverizer at as uniform a rate as possible. Fig. 2 shows the machine in service at the pumping station. After the screenings were ground, they discharged from the pulverizer on the up-stream side

of the screens. At first, it was thought that the ground material would clog the screens, especially after they had become partly covered with unground material, but such was apparently not the case.

Table 2 gives the quantity and the moisture of the screenings removed from the sewage, and Table 3 the quantity of material pulverized and the power used. The quantity of screenings collected and pulverized at the pumping station (77.94 cu. ft. per hour) was considerably more than at the treatment works (62.71 cu. ft. per hour). The reason for this difference is that all the screenings were ground over one period of time at the treatment works, while at the pumping station a portion of the screenings was put into the grinder every half hour. By working intermittently the laborer who did the work had an opportunity to rest, so that the rate at which he put the screenings into the pulverizer was faster than when they were ground over one continuous period. At both places the quantity of current was about ¼ kw-hr. per cu. ft. of material used.

No estimates were prepared of the cost of grinding screenings at the pumping station as compared with pressing and incinerating them. It was apparent that it would not be economical to adopt the grinding method. The cost of coal at the station is cheaper, and the cost of electric current greater than at the sewage works. Furthermore, the cost of grinding the screenings would not be any less because of a saving in floor space as is the case at the treatment works. The tests at the pumping station demonstrated that the ground material did not clog the cage screens, and had no apparent effect on the operation of the pumping equipment.

Discussion of Tests

The experiments showed that the pulverizer would grind sewage screenings satisfactorily with the exception of any large objects that might clog it. A grinder is simple and sturdy, and can be built for rough use. Care must be exercised not to grind excessively large quantities of material at any one time, or the pulverizer will become choked. During the short period in which the experiments were conducted, the introduction of the ground screenings into the raw sewage flowing to the sedimentation tanks had no noticeable effect on the operation of the tanks. Unsatisfactory conditions might arise if the material were put into Imhoff tanks, the sludge digestion compartments of which were too small for the quantity of settling solids to be digested. An examination of the pumps at the pumping station showed no evidence of material being caught under the valve seats to any greater extent than when the screenings were removed from the sewage and incinerated.

At the sewage works it would be slightly more expensive to grind the screenings than to press and incinerate them. At other localities prevailing prices for labor, materials, coal, etc., might be such that the cost of grinding would be less than pressing and incinerating the screenings. It should be emphasized that the digestion space assumed to be necessary at Baltimore is from 50 to 100% more than at many other sewage works, due to the excessive acid condition of the sludge. Therefore, the fixed cost of disposing of the ground screenings is somewhat more than it would be otherwise.

TABLE 2.—QUANTITY OF SCREENINGS COLLECTED, PRESSED, AND INCINERATED, EASTERN AVENUE SEWAGE PUMPING STATION, BALTIMORE, MD.

Date, 1926.	Sewage flow, in million gallons.	Unpressed Screenings.					Pressed Screenings.							Percentage of reduction in volume by pressing screenings.
		Volume.		Weight.		Moisture.	Volume.		Weight.		Moisture.	Coal Used for Incineration.		
		Cubic feet.	Cubic feet per million gallons of sewage.	Pounds.	Pounds per cubic foot.	Percentage.	Cubic feet.	Cubic feet per million gallons of sewage.	Pounds.	Pounds per cubic foot.	Percentage.	Pounds.	Pounds per cubic foot of screenings.	
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)
August 9	22.40	182.25	8.14	10 508.50	57.66	.....	103.60	4.63	4 870.80	47.01	.....	978	9.44	43.16
August 10	23.10	158.50	6.86	8 943.50	56.43	.....	91.95	3.98	3 889.50	42.30	.....	999	10.86	41.99
August 11	22.46	175.00	7.79	10 418.50	59.53	.....	100.75	4.49	4 802.50	47.67	.....	1 061	10.53	42.43
August 12	22.86	158.75	6.94	9 346.00	58.87	.....	79.00	3.46	2 911.50	36.85	.....	897	11.35	50.24
August 13	23.27	189.25	8.13	11 199.50	59.18	86.76	95.50	4.10	4 252.50	44.53	65.10	893	9.35	49.54
August 14	21.56	169.00	7.84	9 702.30	57.41	89.78	96.25	4.46	4 436.50	46.09	77.69	910	9.45	43.05
August 16	19.58	141.50	7.23	8 594.50	60.74	.....	74.45	3.80	3 306.00	44.41	.....	882	11.85	47.39
August 17	22.01	177.45	8.06	11 443.50	64.49	87.65	89.85	4.08	5 091.50	56.67	70.97	1 076	11.98	49.37
August 18	21.98	169.75	7.72	10 362.50	61.05	86.40	77.25	3.51	4 507.00	58.34	69.94	900	11.65	54.49
August 19	20.25	183.25	9.05	11 551.50	63.04	84.02	87.00	4.30	5 970.50	68.63	76.79	1 110	12.76	52.52
August 20	20.51	164.75	8.03	10 605.00	64.37	82.92	85.00	4.14	4 620.50	54.36	66.14	1 000	11.76	48.41
August 21	20.88	151.75	7.29	8 938.50	58.90	87.61	73.55	3.53	4 082.50	55.51	78.72	965	13.12	51.53
Average	.....	.....	7.75	.....	60.17	86.45	.....	4.04	.....	50.03	72.19	972.6	11.07	47.85

TABLE 3.—QUANTITY OF SCREENINGS COLLECTED, PULVERIZED, AND CURRENT USED, EASTERN AVENUE SEWAGE PUMPING STATION, BALTIMORE, MD.*

Date, 1926.	Sewage flow, in million gallons.	Screenings Collected.					Screenings Pulverized.				Current Used.		
		Volume.		Weight.		Moisture.	Time Required to Pulverize.		Cubic feet per hour.	Pounds per hour.	Total quantity, in kilowatt-hours.	Kilowatt-hour per cubic foot.	Kilowatt-hours per 1 000 lb.
		Cubic feet.	Cubic feet per million gallons.	Total, in pounds.	Pounds per cubic foot.	Percentage.	Hours.	Minutes.					
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)		(9)	(10)	(11)	(12)	(13)
September 14...	22.38	215	9.61	10 897	50.7	79.35	2	40	80.52	4 081	60	0.279	5.51
September 15...	30.18	260	8.61	13 653	52.5	87.12¶	3	07	83.33	4 376	67	0.258	4.91
September 16...	30.05	224†	7.45	12 620†	56.3	84.16¶	2	36	78.46	4 446	54	0.265	4.67
September 17...	31.04	228‡	7.35	12 411‡	54.4	83.82¶	1	31	94.74	4 997	34	0.236	4.48
September 18...	27.60	202	7.32	10 787	53.4	.....	2	54	69.66	3 720	50	0.248	4.64
September 20...	31.20	269	8.62	14 226	52.9	86.96¶	2	59	90.27	4 774	52	0.193	3.66
September 21...	30.86	290	9.40	15 022	51.8	84.85¶	3	20	87.09	4 511	69	0.238	4.59
September 22...	31.70	226	7.13	11 824	52.3	86.55¶	2	52	78.75	4 120	58	0.257	4.91
September 23...	31.41	306	9.74	15 922	52.0	85.95¶	4	29	68.30	3 554	83	0.271	5.21
September 24...	32.25	298	9.24	15 463	51.9	87.15¶	4	08	72.16	3 744	70	0.235	4.53
September 25...	31.05	292	9.40	15 149	51.9	81.61¶	3	48	76.84	3 987	71	0.243	4.69
September 27...	27.17	284	10.45	15 041	53.0	78.83	3	57	71.90	3 808	74	0.261	4.92
September 28...	31.15	238	7.64	12 555	52.7	88.13¶	3	18	72.12	3 805	67	0.282	5.34
September 29...	30.43	270	8.87	14 166	52.5	73.35	3	39	78.97	3 881	73	0.270	5.15
September 30...	29.82	284	9.52	14 848	52.3	87.23	3	10	89.59	4 684	72	0.254	4.85
October 1.....	29.57	258	8.73	13 631	52.8	84.21	3	26	75.22	3 974	74	0.287	5.43
October 2.....	30.17	260	8.62	13 769	53.0	85.28	3	22	77.15	4 086	68	0.262	4.94
October 4.....	31.36	268	8.55	14 078	52.5	87.97	3	20	80.48	4 228	66	0.246	4.69
October 5.....	32.87	274	8.34	14 406	52.6	88.46	3	54	70.26	3 694	74	0.270	5.14
October 6.....	32.71	300§	9.17	15 806§	52.7	81.12	3	01	84.11	4 444	60	0.236	4.47
October 7.....	29.78	290‖	9.74	15 071‖	52.0	82.72	2	21	82.55	4 274	46	0.237	4.58
Average.....	.....	....	8.72	.....	52.63	84.70	..	....	77.94	4 097	....	0.254	4.83

* Screenings collected each day over 24-hour period except September 14, when they were collected for 16½ hours. All screenings were pulverized except on September 16 and 17, and October 6 and 7.

† 204 cu. ft. (11 559 lb.) pulverized.

‡ 144 cu. ft. (7 596 lb.) pulverized.

§ 254 cu. ft. (13 422 lb.) pulverized.

‖ 194 cu. ft. (10 043 lb.) pulverized.

¶ Average of two analyses.

Disadvantages of Grinding Screenings.—The process of grinding the screenings and returning them again to the sewage naturally has a limited applicability. At many sewage works considerable effort is made to remove as much coarse material as possible, partly to lighten the load on subsequent treatment units. Under such circumstances the adding of ground screenings to the sewage would defeat the purpose for which the screens were intended. It is difficult to predict whether this material would increase the formation of scum on settling tanks noticeably, but that is possible. At some places where screenings could not be disposed of temporarily, two pulverizers might be necessary—one to be used as a spare unit. In such cases, the cost of grinding would be slightly increased.

Advantages of Grinding Screenings.—At the Baltimore sewage works one of the advantages that the grinding of screenings has over pressing and incinerating is that one relatively small and inexpensive piece of machinery can be used instead of more expensive and elaborate equipment. Every device that can be omitted satisfactorily at a sewage works is conducive of simplicity in operation. Less labor is actually required to grind the screenings than to press and incinerate them. The question of hauling, handling, and providing storage space for coal is eliminated; there are no ashes and clinkers; and the screenings need be out of the sewage only a short time before they are ground and returned. As a result, odor and fly nuisances, which frequently must be considered, should be relatively negligible.

Conclusions

The grinding of sewage screenings in a suitable pulverizer affords a means of disposing of them where local conditions favor such a method. The process should offer no great difficulties because many other materials of a considerably more abrasive and resistant nature have been successively ground at manufacturing plants and similar establishments for a number of years. During the tests no difficulties were experienced. The chief factor that increases the cost of the process (which should not be overlooked) is the disposition of the screenings in the form of sludge. It is usually recognized, however, that a well-digested sludge presents fewer difficulties in handling than sewage screenings. Like other innovations in sewage treatment, the methods of pulverizing screenings should be considered with caution, and all the factors involved should be studied carefully before the process is adopted.

Acknowledgments

Full credit should be given to John H. Gregory, M. Am. Soc. C. E., Consulting Engineer to the Bureau of Sewers of Baltimore, for calling the writer's attention to the possibilities connected with the grinding of sewage screenings. It was chiefly through the former's efforts that the manufacturer of the pulverizer agreed to send it to Baltimore so that a series of tests could be made.

AMERICAN SOCIETY OF CIVIL ENGINEERS
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PAPERS AND DISCUSSIONS

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FRICTION OF SEWAGE SLUDGE IN PIPES

PROGRESS REPORT OF COMMITTEE OF THE SANITARY ENGINEERING DIVISION*

When the Committee was appointed, it was thought that the study of friction of sewage sludge in pipes would require comparatively little time. However, the subject has proved to be involved, and somewhat elusive as to conclusions. The Committee presents the following statement as a progress report, summarizing its investigation to date (1928).

Historical Summary

Until recently definite data on the flow of sewage sludge through pipes have been lacking. Only a few references have been found describing data compiled prior to 1913. A German custom was to allow a sludge friction head about six times as much as that for water. On the other hand, at Birmingham, England, the formula for water in clean cast-iron pipes was considered applicable for septic sludge of approximately 90% moisture. In 1913, at Elberfeld, Germany, 1.6 atmospheres of pressure were required to pump sludge with a 92% moisture content through an 8-in. pipe, 2 850 ft. long (approximately 18.25 ft. per 1 000 ft.; quantity not known).

Mr. W. H. Lindley, at Frankfort-on-the-Main, Germany, first suggested a formula, although he notes that the friction will vary with the character of the suspended matter in the sludge and the water content. Sludge at Frankfort was handled with water contents ranging upward from 85 per cent.

Let J = hydraulic grade.
V = velocity, in meters per second.
R = hydraulic radius, in meters.
C = coefficient, approximately 0.0007.

Then, the formula suggested by Mr. Lindley is:

$$J = C \frac{V^{1.8}}{R^{1.25}} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (1)$$

He considers this to be a rough rule and advises checking by special tests if the case is important.

Note.—Written discussion on the report will be closed in December, 1929.

* Presented at the meeting of the Sanitary Engineering Division, New York, N. Y., January 19, 1928.

For comparison he indicates the following values for the coefficient, C, for water and sewage:

Pure water in cast-iron pipes	0.00018
Pure water in concrete pipes	0.00015 to 0.00020
Pure water in brick conduits	0.00020
Sewage in brick sewers	0.00025
Sludge	0.0007

These values are higher than those for new pipes or conduits. In the friction of sludge, the quantity of grit is important. It should be reduced as much as possible for long pipe lines.

Other eminent engineers have estimated sludge friction from two to ten times that of water, the character and density of the sludge being governing factors. The Committee has failed to locate any actual tests made prior to 1913. Engineers confronted with a sludge friction problem have used a liberal factor of safety.

English Experience.—In 1923, Mr. George A. Hart, at Leeds, England, noted than when sludge flows through a pipe line at a velocity of about 6 ft. per sec. there is little difference in the friction of sludge with 90% moisture content and the friction of water. Hart believed that difficulties in pumping sludge resulted from deposits of solids during quiescent periods.

In the same year, Mr. H. C. Whitehead, at Birmingham, England, noted that sludge with a 95% moisture content had perhaps three times the friction of water and with 88 to 90% moisture content, twelve times the friction of water. The practical need of avoiding deposits or caking of solids was emphasized.

In 1924, Mr. William E. Clifford* showed that the laws of flow which hold for homogeneous fluids may not hold for sewage sludge. The discussion, however, attempted to deal with the subject on a viscosity basis and, therefore, as noted hereafter, is subject to error.

In 1925, tests made at Leeds were published† in a diagram. The curves were drawn for velocities of 6 ft. per sec., or more. They were derived from the averages of tests ranging from 89 to 99% moisture, and no knowledge is at hand as to how the tests were made, what the actual results were, or how the testing apparatus was arranged.

In 1926, Mr. Whitehead reported pumping sludge containing 90% of water at velocities of 1.5 to 1.7 ft. per sec. At this velocity, the friction head with digested Birmingham sludge is 4.5 times that of clean water.

Canadian Experience.—In 1919, Mr. T. H. Nevitt made some tests at Toronto, Ont., Canada, on the flow of sludge in a 12-in. pipe, from which he concluded that the value of n in Kutter's formula ranged from 0.0168 to 0.0181. Sedimentation tank sludge not more than 10 days old was delivered by a centrifugal pump. The temperature was 54° Fahr. and the moisture content, 95.9 per cent. All the tests were made with velocities ranging between 5.73 and 6.36 ft. per sec. The Hazen-Williams value of C ranged from 108 to 122.

* *Transactions*, International Conference on San. Eng., July 7, 1924, p. 59.
† *Contractor's Record and Municipal Engineering*, November 4, 1925, p. 1617.

American Experience.—In the design of the sewage works at Baltimore, Md., E. B. Whitman, M. Am. Soc. C. E., made some tests (1908) to determine the friction of sludge in a centrifugal pump. A propeller was set up and operated by a weight, connected over a pulley. The time was noted for a given drop of the weight when the propeller was in sludge and also when in water; the drop of the weight caused 85 revolutions of the propeller in each case. The resistance of sludge was found to be greater than that of water. With the propeller immersed in water, the weight took 15 sec. to drop; in settled sludge, 21.25 sec.; and in septic sludge, 23.8 sec.

In December, 1922, Mr. J. X. Cohen ran a series of tests under adverse climatic conditions, at Toronto. It was probably due to the weather that these tests failed to show any conclusive results.

In 1923, at Baltimore, tests on sludge from a separate sewerage system showed rather erratic results because of the variations of the sludge. The results indicated that the losses of head are affected only slightly by moisture content for percentages exceeding 96, and are increased quite rapidly when the solids increase to more than 5 per cent.

In 1926, at Syracuse, N. Y., Glenn D. Holmes, M. Am. Soc. C. E., noted that in a 5-in. line, 12 000 ft. long, an average velocity of 2.5 ft. per sec. was maintained, but no trouble was found with velocities as low as 1 ft. per sec. for considerable periods. This was on sludge from a combined sewerage system produced by 1-hour settling. After rain storms, during which larger quantities of sand were brought to the plant, the velocity was increased to 4 ft. per sec. for a short period, to insure flushing. The sludge averaged 99.5% moisture. In an attempt to pump 90% sludge, the pressures increased so rapidly that the test was abandoned.

Laws of Flow

The flow of water, as ordinarily discussed, is a particular case of the flow of fluids in general. The Committee is especially indebted to two sources* for the general discussion immediately following.

At low velocities every fluid flows in straight-line motion, with no mixing currents. The equation of flow is given by Poiseuille's law as:

$$p = \frac{0.000668\, z\, l\, v}{D^2} \qquad (2)$$

in which,

p = pressure drop, in pounds per square inch.
z = viscosity in centipoises (relative to water at 68° Fahr.).
l = length of pipe, in feet.
v = velocity in feet per second.
D = inside diameter of pipe, in inches.
f = friction factor which varies with the modulus, $\frac{D\, v\, s}{z}$.

"The term, absolute viscosity, which appears in the above equation [Equation (2)] is a measure of the internal fluid friction. Each fluid has a definite

* "Principles of Chemical Engineering," by Messrs. W. H. Walker, W. K. Lewis, and W. H. McAdams, N. Y., McGraw-Hill Pub. Co., 1928, pp. 73–88; and *Journal of Industrial and Engineering Chemistry*, Vol. 14, February, 1922, pp. 105–119.

viscosity at a given temperature, and the absolute viscosity of fluids varies with temperature."*

As the velocity is increased, some point is reached at which the type of motion suddenly changes from straight line to turbulent motion. The equation of flow for turbulent motion is given typically by Fanning's equation:

$$p = \frac{0.323\, f\, l\, s\, v^2}{D} \qquad (3)$$

In Equation (3), f and the exponents of v and D, are varied by different authorities to fit test data. For a wide range of conditions f is taken as a function of $\frac{D\, v\, s}{z}$.

"The velocity at which the type of motion changes from straight line to turbulent flow is called the critical velocity",† which may vary widely for different fluids. "The critical velocity is large for large absolute viscosity, small fluid density, and small inside pipe diameter."‡ Table 1§ shows these variations.

TABLE 1.—Calculated Critical Velocities in Standard 2-Inch Pipe.

Fluid.	Temperature, in degrees Fahrenheit.	Absolute pressure, in atmospheres.	Density, in pounds per cubic foot.	Absolute viscosity, in English units.	Critical velocity, in feet per second.
Hydrogen	70	1	0.00522	0.0000062	6.5
Air	70	1	0.0750	0.0000125	0.91
Air	70	10	0.750	0.0000125	0.091
Steam	212	1	0.0373	0.0000081	1.19
Steam	357	10	0.326	0.0000097	0.16
Water	68	any	62.3	0.000672	0.059
Refined oil	68	any	56.0	0.0007	0.65
California crude oil	68	any	60.0	2.32	211

The critical velocities of air and water are so low compared with those in the usual pipe lines that straight-line flow is seldom found in practice for these fluids except in small pipes.

The viscosity is usually determined by a viscosimeter. The more common form of this instrument consists of a small cylindrical reservoir with a conical bottom to which a short efflux tube is attached. The instrument is calibrated by noting the time of efflux for liquids of known viscosity. "In the case of liquids the viscosity always decreases as the temperature rises."‖

In addition to the complications which occur in attempting to derive a formula for a reasonably homogeneous fluid, the Committee would point out the variable character of sludge, and would suggest that its true character is not that of a homogeneous fluid, but a mass of matter in suspension, or a mixture. It is evident from the tests made by the Committee and from

* "Principles of Chemical Engineering," by Messrs. W. H. Walker, W. K. Lewis, and W. H. McAdams, p. 74.
† *Loc. cit.*, p. 75.
‡ *Loc. cit.*, p. 76.
§ *Loc. cit.*, p. 77.
‖ *Loc. cit.*, p. 80.

attempts to derive coefficients of friction and viscosity that the laws for viscous flow are not applicable.

Behavior of Mixture.—For comparison Fig. 1 has been plotted for mixtures of filter sand and water in a 4-in. pipe, from data given by Allen Hazen, M. Am. Soc. C. E.* These curves for different percentages of solids (by weight) resemble the test curves plotted from data on friction of sewage sludge obtained at Chicago, Ill.

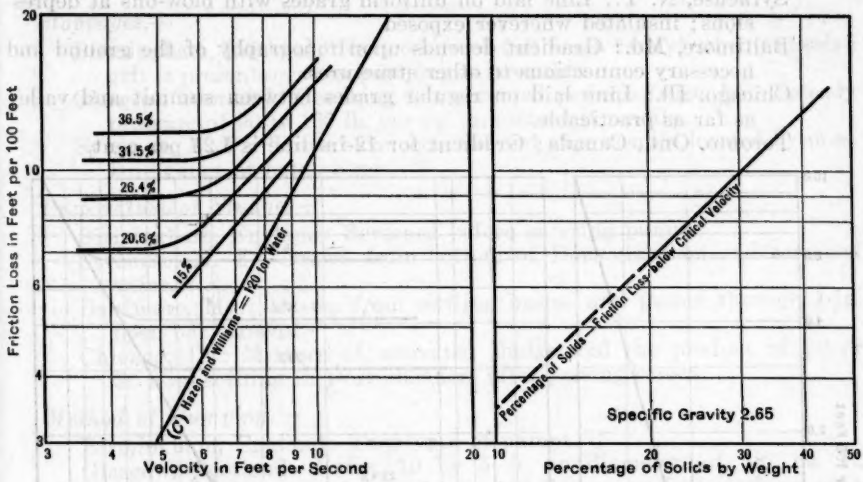

FIG. 1.—FRICTION LOSS OF FILTER SAND AND WATER IN 4-INCH PIPE.

Furthermore, Fig. 2 was plotted for mixtures of clay slurry in a 4-in. pipe, from data given by W. B. Gregory, M. Am. Soc. C. E.† These curves for different percentages of solids resemble the test data obtained at Syracuse, N. Y. The dotted line, marked 23.6%, was obtained with material much finer than the remainder. Its specific gravity was 2.38.

Other data are available on sand and water mixtures in small pipes‡ and also on pulp and water mixtures.§ Such studies seem to indicate that sewage sludge friction varies in a manner similar to that for fluids with material in suspension and not in accordance with the laws for viscous fluids.

EXISTING PIPE LINES

Most of the long pipe lines for sludge are to be found in Great Britain. The characteristics have been summarized by Mr. William E. Clifford.‖ The Leeds line has been built since his table was prepared, and no description is available at present. The following comments and descriptions have been taken from Clifford, with certain additions by the Committee:

* American Engineer's Handbook, p. 1217.

† *The Mechanical Engineer*, June, 1927, p. 609.

‡ *Transactions*, Am. Soc. C. E., Vol. LVII (1906), p. 400.

§ Series of tests made by Mr. W. J. Trumby for the International Paper Co., diagrams in "Manufacture of Pulp and Paper," Vol. V, p. 67. Tables also pub. by Guild's Pump Co., Seneca Falls, N. Y.

‖ *Transactions*, International Conference on San. Eng., July 7, 1924, p. 59.

Line and Grade.—

Birmingham, England: Not laid on a grade; inlet and outlet practically level.

Glasgow, Scotland: Practically level.

Bradford, England: Line laid on a grade with an open end for free discharge.

Wolverhampton, England: Pipes laid on surface of ground without a regular gradient.

Syracuse, N. Y.: Line laid on uniform grades with blow-offs at depressions; insulated wherever exposed.

Baltimore, Md.: Gradient depends upon topography of the ground and necessary connections to other structures.

Chicago, Ill.: Line laid on regular grades between summit and valley as far as practicable.

Toronto, Ont., Canada: Gradient for 12-in. line is 1.25 per cent.

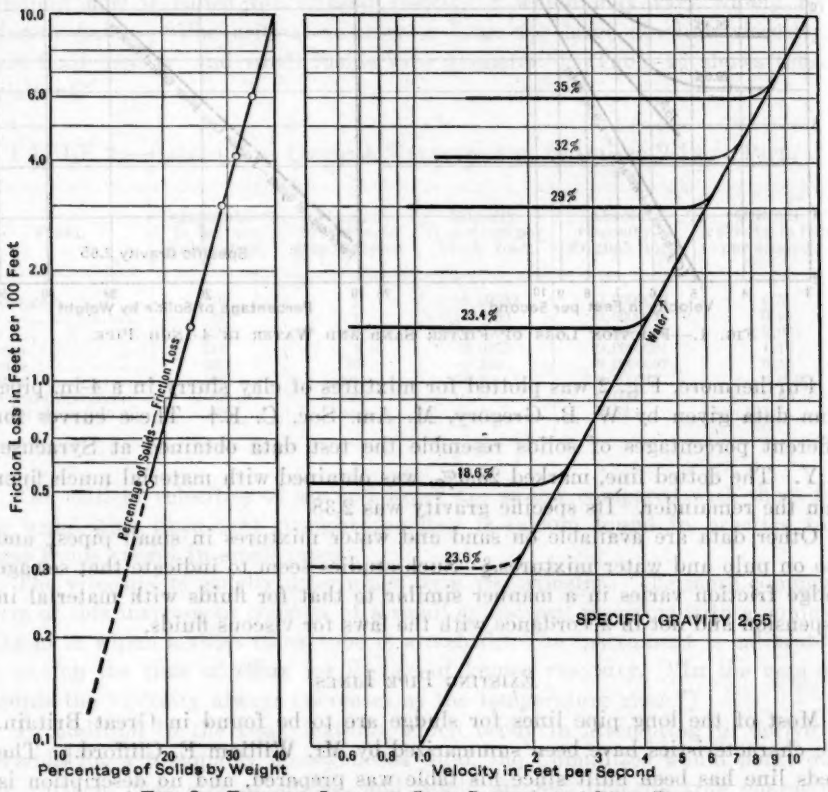

FIG. 2.—FRICTION LOSS OF CLAY SLURRY IN 4-INCH PIPE.

Maintenance.—

Birmingham, England: Access boxes and provisions for flushing under pressure are provided, but have not been needed; line flushed twice each year by pumping sewage at 3 or 4 ft. per sec. for 12 hours.

Glasgow, Scotland: Two scour valves at excessive dips are tested regularly; after pumping sludge of 88% moisture, a thinner sludge is used as a precaution.

Bradford, England: When sludge pumping is stopped a few days the line is blown out with compressed air.
Syracuse, N. Y.: Manholes with hatch-boxes every 500 ft.
Chicago, Ill.: Manholes with hatch-boxes every 3 260 ft., more or less.

Air Valves.—

Birmingham, England: Provision for valves at intermediate summits, but valves removed.
Chicago, Ill.: At all summits.

Stoppages.—

Birmingham, England: Have occurred (although rarely) when excessive grit is present or when pumping is irregular.
Glasgow, Scotland: Checking has occurred perhaps twice, but a steady pressure of 90 to 100 lb. per sq. in. cleared it readily.
Syracuse, N. Y.: Three have occurred and were readily located by pressure gauge at hatch-boxes.

Preparation of Sludge.—

Birmingham, England: Screened before entering pumps.
Syracuse, N. Y.: Drawn from bottom of Dorr clarifiers and screened twice.
Baltimore, Md.: Drawn from settling basins and passed through ¾-in. open bar screens.
Chicago, Ill.: Mixture of activated sludge and the product of 20 or 30 min. settling in Dorr clarifiers after passing screen.

Method of Pumping.—

Birmingham, England: Two types of pumps.
Glasgow, Scotland: Rams, 10 by 5 ft. in diameter, of 187 cu. ft. capacity; rate, 1 ton per min. at 100 lb. per sq. in.; working pressures from 60 to 80 lb. per sq. in.
Bradford, England: Plugs of sludge are followed by compressed air at 50 to 60 lb. per sq. in.
Wolverhampton, England: Compressed air.
Syracuse, N. Y.: Four 6 by 10-in. stuff pumps; 5-in. bronze ball-valves; 15-h.p., slip-ring motors; capacity, 92 gal. per min. against 462-ft. head; two pumps for ordinary service at 80-ft. head.
Baltimore, Md.: Centrifugal pumps.
Chicago, Ill.: Centrifugal pumps of special design, to pass solids and keep runners clean; capacity, each 1 000 gal. per min. against 188 ft. dynamic head; 100-h.p. squirrel-cage motors at 1 765 rev. per min.
Toronto, Ont., Canada: 18-in. centrifugal pump through a 44-ft. lift to the entrance of the line.

Miscellaneous.—

Birmingham, England: No trouble has been experienced at the joints.
Glasgow, Scotland: Longest line now in existence (as of 1928).
Bradford, England: Numerous summits tend to break up the sludge and air pockets.
Syracuse, N. Y.: Longest finished pipe line in the United States (as of 1928).
Baltimore, Md.: This is one of several lines; the longest is more than 5 000 ft.
Chicago, Ill.: Pipe is shop tested to 300 lb. per sq. in. and tested in place to 175 lb. per sq. in.
Toronto, Ont., Canada: In addition, another line of 400 ft. of 10-in. spiral riveted pipe was used in the 1919 and 1929 tests.

Tests Collated from Others

Toronto, Ont., Canada, Tests.—In December, 1922, Mr. J. X. Cohen made some tests at Toronto on a 12-in. pipe, with a test section of 240.5 ft. in a 400-ft. run; ¼-in. piezometer connections were used, with water columns. Fresh sludge with a moisture content ranging from 91 to 93% was used. The specific gravity varied from 1.010 to 1.017. The temperature of the sludge ranged from 42 to 46° Fahr., but the air temperatures ranged from 0 to 32° Fahr. The weather conditions were unsatisfactory and freezing often occurred in the piezometers despite the precautions taken. Runs lasted 30 min., with readings every minute. Some surging occurred in the pipe line.

The test arrangements did not provide an opportunity to vary the moisture of the sludge and the temperature. A Venturi meter was used to measure the flow, but no rating was made on sludge. Viscosity determinations made at 68° Fahr. did not help matters. For moistures between 94.5 and 96.1%, the results on one run give a value of C from 78 to 94. On the other runs C varied so widely that no deductions can be drawn. In an endeavor to adjust the viscosity determinations, Mr. Cohen ran the sludge through a meat chopper. This procedure indicates that sludge is not a viscuous material obeying the laws of viscosity alone. It may contain stringy, fibrous material, which must be handled.

The data secured during these tests are not consistent and are unreliable as shown by the divergence of the results. The difficulties imposed by the low atmospheric temperatures were largely responsible for the failure to secure reliable information.

Tests at Baltimore, Md.—The first tests on friction in pipe lines were run at Baltimore in 1923 on an 8-in. cast-iron flanged pipe, with piezometers 136.5 ft. apart. Sludge was delivered by a centrifugal pump. Twenty-five runs were made from 6 to 17 min. long, with velocities from 3 to 6.3 ft. per sec. and moisture ranging from 94 to 99.4 per cent. Some difficulty was experienced in keeping the moisture constant. The conclusion of the Baltimore observers was that, for Kutter's n, a variation with the percentage of moisture, w, was expressed for undigested sludge by the relation, $n = 0.067 - 0.00057\ w$. In the Hazen-Williams formula, $C = 6.79\ w - 551$. For ordinary conditions, corresponding to water ($n = 0.012$ and $C = 100$ in the Hazen-Williams formula), the relations are given as $n = 0.069 - 0.00057\ w$, or $C = 5.79\ w - 579$. The use of a 4-in. piezometer on this 8-in. line has been questioned. Considerable variation in readings occurred, sometimes 6 to 12 in. on a probable 2-ft. loss of head. Others have noted such pulsations from a pump.

As the value of C varied from 118 to 142 for water with a velocity of 7.1 to 8.9 ft. per sec., the results are not recommended by the Committee, but are given to show the steps taken at Baltimore. For sludge, C varied from 154 with 97.1% moisture to 92.8 for 98.5% moisture. Additional tests were made in 1927.

At Baltimore the sewage from a separate system of sewers flows through screens with ⅜-in. openings into sedimentation tanks, two of which are always kept in operation. Each of these tanks has a working capacity of 3 800 000 gal. and provides for the removal of about 85% of the suspended solids. After

a tank has been in service for a week, it is by-passed and unwatered. The sludge is pumped to one or more of fifty-one digestion tanks.

During the tests the sludge was handled by an 8-in. vertical "Wood" trash pump, with a rated capacity of 7 cu. ft. per sec. (3 200 gal. per min.) against a total head of 50 ft., driven by a variable-speed, 75-h.p. motor, running at 1 160 rev. per min. The pump operated at all times under a positive head. The sludge flowed from the settling tank to the pump well in an egg-shaped conduit, 48 by 32 in.; then through 6.5 ft. of 12-in. cast-iron pipe to the suction of the pump.

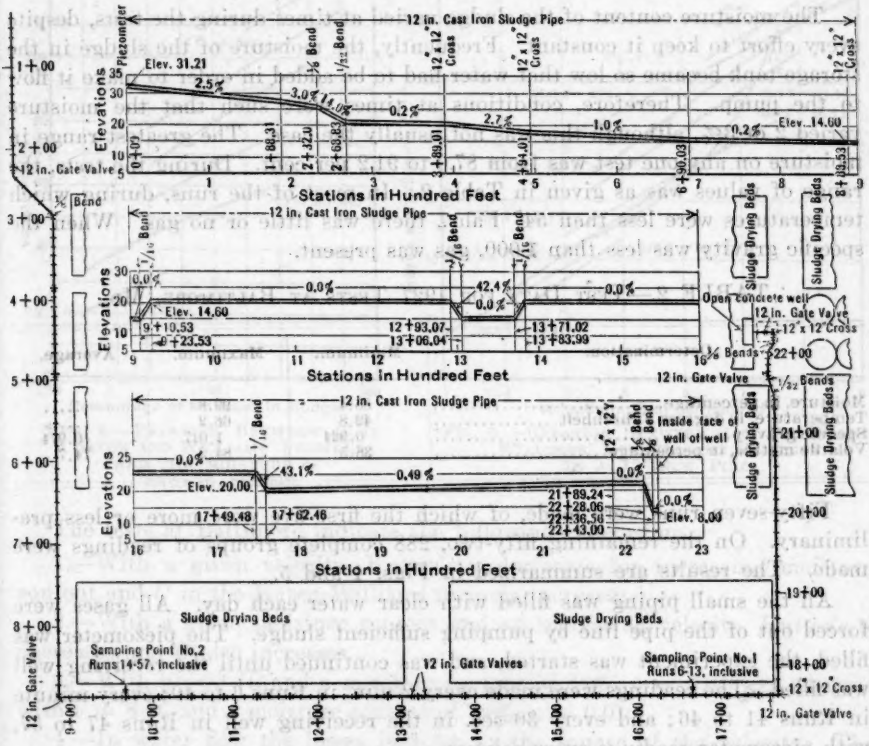

FIG. 3.—PLAN AND PROFILE OF SLUDGE PIPE IN BALTIMORE, MD.

The pipe line (see Fig. 3) is a 12-in., Class A, cast-iron force main, 2 252 ft. long, with bell and spigot joints, except for 39 ft. The pipe terminates in a receiving well, which served for a measuring tank. In the force main are ten gate-valves, three $\frac{1}{32}$ bends, eight $\frac{1}{16}$ bends, and six $\frac{1}{8}$ bends, as well as two tees and two Y branches, twenty-five crosses through which the flow passes in a straight direction, and three crosses through which the flow is diverted 90 degrees. The pipe has been in service since 1912. In 1924 inspection showed it to be in good condition and practically free from tuberculation. The inside diameter is assumed to be 12.12 in. (see Class A pipe, specifications of the American Water Works Association).

The measuring well was 28.5 ft. by 10 ft. in plan and 30 ft. deep. The surface elevation of the sludge was determined by a tape with an 8-in. wooden disk fastened to the lower end, which was floated on top of the sludge.

The losses of head in the pipe line were determined by making simultaneous readings of a piezometer and the sludge in the receiving well. The piezometer was a 4-in. steel pipe, about 15 ft. long, welded to the 12-in. pipe. A valve at the upper end was opened from time to time. A ¼-in. pipe below the gate-valve was connected with a U-tube containing mercury. When the pressure was too great for the piezometer, a 30-lb. pressure gauge was used and read to the nearest ¼ lb.

The moisture content of the sludge varied at times during the tests, despite every effort to keep it constant. Frequently, the moisture of the sludge in the storage tank became so low that water had to be added in order to make it flow to the pump. Therefore, conditions at times were such that the moisture varied 2 or 3%, although this was not usually the case. The greatest range in moisture on any one test was from 87.0 to 91.2 per cent. During the tests, the range of values was as given in Table 2. In most of the runs, during which temperatures were less than 54° Fahr., there was little or no gas. When the specific gravity was less than 1 000, gas was present.

TABLE 2.—TEST DATA FOR 1927 TESTS AT BALTIMORE, MD.

Determination.	Minimum.	Maximum.	Average.
Moisture, in percentage	85.4	99.8	.....
Temperature, in degrees Fahrenheit	42.8	66.2	.....
Specific gravity	0.924	1.017	0.974
Volatile matter, in percentage	38.5	84.2	74.7

Fifty-seven runs were made, of which the first five were more or less preliminary. On the remaining fifty-two, 288 complete groups of readings were made. The results are summarized in Figs. 4 and 5.

All the small piping was filled with clear water each day. All gases were forced out of the pipe line by pumping sufficient sludge. The piezometer was filled, the experiment was started, and was continued until the receiving well was filled. The readings were made every 2 min. in Runs 6 to 40; every minute in Runs 41 to 46; and every 30 sec. in the receiving well in Runs 47 to 57, with piezometer readings every minute.

Sludge samples were taken every 2 min. by opening quickly a 10-in. gate-valve. The sampling point was changed after the thirteenth run to Point No. 2 (see Fig. 3). The valve was opened after reading the piezometer. Samples of 1 gal. were taken, 1 pint of which was used for analysis. These were composited in some tests.

To keep the head constant, as the receiving well filled, a partly closed gate-valve near the pump was opened gradually. A 60-lb. pressure gauge was used to show the pressures. The receiving well was filled to within 3 ft. of the top and then pumped out to within 8 ft. of the bottom. From one to seven runs were made daily. Close attention was paid to the piezometer to keep it from clogging. Gas accumulation was also watched.

The results are somewhat disappointing, so far as uniformity is concerned. Considering the difficulties of the test layout and the nature of the sludge, they indicate the variations to be met in handling such material. The calculations are based on the straight line, without allowance for bends, crosses, etc.

The average moisture of the sludge in the line was computed from the sampling and known velocities. The plots did not indicate any data which point to a definite value of C or n for any given moisture or velocity. Large variations are found between different tests.

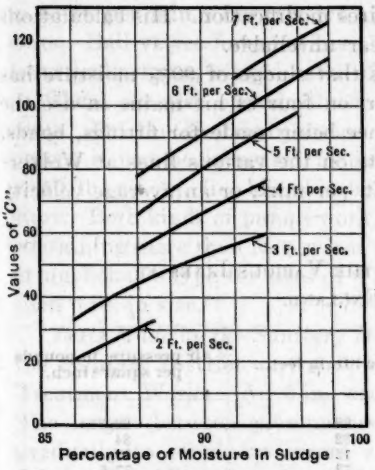

FIG. 4.—RELATION BETWEEN C IN HAZEN AND WILLIAMS' FORMULA AND THE MOISTURE OF SEWAGE SLUDGE.

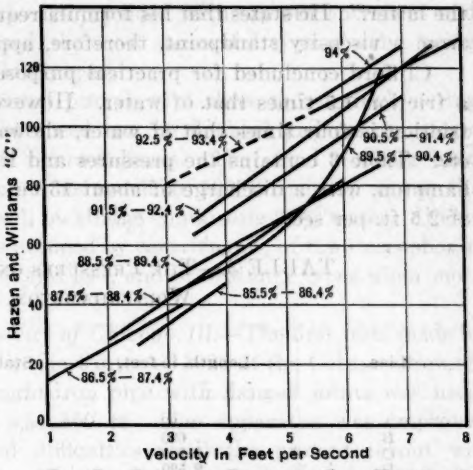

FIG. 5.—RELATION BETWEEN C IN HAZEN AND WILLIAMS' FORMULA AND VELOCITY IN A 12-INCH PIPE.

The tests at Baltimore indicate the following conclusions:

1.—With a given velocity, Kutter's n decreases for increasing moisture content and C in the Hazen-Williams formula increases.

2.—With a given moisture content and an increasing velocity, Kutter's n decreases and C also increases.

3.—With $v = 3$ ft. and a moisture content of 88%, $n = 0.020 \pm$, whereas with $v = 8$ ft. and a moisture content of 92%, $n = 0.010$.

4.—In water flow the losses increase as the square of the velocity. The Baltimore tests indicate an increase of about 50% on sludge when the velocity is doubled.

Wolverhampton, England, Tests.—Some tests were made by Mr. William E. Clifford on 4-in. wrought-iron pipe with conical socket and spigot joints laid on the surface of the ground.* Compressed air was the motive force, and the velocities averaged about 2.5 ft. per sec. Clifford concluded the average friction head to be 24.5 ft. per 1 000 ft., but he does not give either moisture or temperature. He attempted to analyze sludge as a viscous fluid, using a viscosimeter modified after the Cohen instrument, through which the sludge would

* "Friction of Sewage Sludge in Pipes", by W. E. Clifford, International Conference on San. Eng., 1924, pp. 59-65.

not pass until it had been minced in a meat chopper, thus destroying its physical characteristics.

Furthermore, he states that "it does not follow that the law which holds for homogeneous fluids will hold for sewage sludge."* Using data derived from the 4-in. pipe and a kinematic viscosity obtained from apparently unrelated data, he calculates certain friction heads for sludge on sizes of pipe from 4 to 12 in., with velocities from 2.5 to 5.0 ft. per sec. He adds, "A comparison of the friction losses with water in pipe lines with that obtained from the kinematic viscosity relationship indicates that the former is more than twice the latter." He states that his formula requires modification. His calculations from a viscosity standpoint, therefore, appear unreliable.

Clifford concluded for practical purposes that sludge of 90% moisture has a friction 1.5 times that of water. However, on four of his mains in use the friction is four times that of water, allowance being made for fittings, bends, etc. Table 3 contains the pressures and lifts on the various lines at Wolverhampton, with a discharge of about 13 cu. ft. per min., or an average velocity of 2.5 ft. per sec.

TABLE 3.—THE PRESSURES ON THE VARIOUS LINES AT WOLVERHAMPTON, ENGLAND.

Line.	Length, in feet.	Static lift, in feet.	Air pressure, in pounds per square inch.
A	99	22	22.5
B	690	32	34
C	3 900	12	55
D	2 520	73	77.5
E	1 770	55	56
F	2 700	56	67.5
G	3 750	78	85

Leeds, England, Tests.—At Leeds, pumping is done by two horizontal double-acting pumps, each belt-driven by a 50-h.p. motor, with a capacity of 36 000 gal. per hour. The pumps have 10-in. cylinders, 18-in. stroke, 12-in. suction, and 9-in. discharge. The experiments at the low-level works at Leeds are not described, but the conclusions given are of interest.† Tests were made on sludge with moisture contents of 89.4 to 99%, on a 6-in. pipe. The conclusions were:

(1) Viscosity had no practical bearing on the problem.
(2) Flow of sludge followed the same law as water for all velocities greater than the self-cleansing velocity of sludge.
(3) The self-cleansing velocity of sludge at Leeds was 6 ft. per sec.
(4) With velocities less than 6 ft. per sec., the flow varied according to no law, but was influenced by the amount of deposits.
(5) The formula derived for cast-iron pipe, at velocities of more than 6 ft. per sec., was found to be as follows:

$$\frac{H}{L} = 0.000058 \frac{V^{1.866}}{R^{1.2}} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (4)$$

* "Friction of Sewage Sludge in Pipes", by William E. Clifford, *Transactions*, International Conference on San. Eng., 1924, p. 60.

† *Contractor's Record and Municipal Engineering*, November 4, 1925, p. 1617.

in which,

H = loss of head due to friction, in feet.
L = length of pipe, in feet.
V = velocity, in feet per second.
R = hydraulic mean depth, in feet.

Equation (4) is submitted by the Leeds observers to permit a comparison with water.

Method of Pumping.—At Birmingham, England, two types of pumps are used. One is a direct-acting duplex, steam-driven, with a piston speed limited to 45 ft. per min. Large valve areas are provided with smooth entrances and exits. Ball valves have not been used, the most successful type of valve being a vertically hung, hinged valve of large area.

There are also electrically operated pumps with three-throw single-acting rams. On the electrical pumps the speed can be doubled and on the steam pumps it can be increased to meet reduced heads. The usual working head is 50 lb. per sq. in., but this becomes as much as 100 lb. per sq. in. at the start. Both kinds of pumps work well on sludge containing 90% water. Sludge containing more than 95% water is pumped by centrifugals, of the stereophagus or unchokable type for sizes of 6 in. and less, and of ordinary types when more than 6 in. in size.*

Tests Made by the Sanitary District of Chicago, Ill.—The first tests made by the Sanitary District were started in the winter of 1925 at the Calumet Sewage Treatment Works. An 8-in. wrought-iron pipe with flanged joints was used. The length between piezometers was 100 ft. The apparatus was improved gradually to eliminate errors and difficulties until the present layout was developed. All the results used in this discussion were derived in 1927 with the final equipment.

The apparatus is set up alongside two rectangular concrete tanks, each 14 by 14 ft. in plan and 14.5 ft. deep, inside dimensions, originally provided for the sludge conditioning at the works. To one of the tanks, the sludge is piped from the Imhoff tanks, the sludge being elevated originally from the initial tank by an air-lift. Filtros plates were provided in the bottom for agitation.

A centrifugal pump was set in a pit alongside the supply tank to receive the sludge and force it through an 8-in. pipe into a stand-pipe, 3 ft. in inside diameter and 15 ft. high, provided with an overflow back to the suction tank. The sludge then flows along one side of the piping loop, around two bends, and enters on the straightaway for the test (Fig. 6). The piezometers were set 100 ft. apart. The one at the entrance is 11 ft. 8 in. from the nearest bend, along the direction of flow.

At the discharge end a 10-in. riser is provided behind a gate-valve to remove any gas which may accumulate. The flow discharges into a stilling-box, 27 in. wide and 16 ft. long, with an 18-in., sharp-edged, rectangular weir at the far end (Fig. 6). The flow depth over the weir is measured by a gauge set 6 ft. back from the crest. The discharge drops into the second concrete tank and is measured volumetrically for calibration of the weir. The pump

* "The Works of Birmingham, Tame and Rea District Drainage Board", by H. C. Whitehead, January, 1926, pp. 33-34.

can draw from either of the two tanks. The same sludge could be used many times by utilizing the two tanks, or its moisture content could be adjusted by dilution with water.

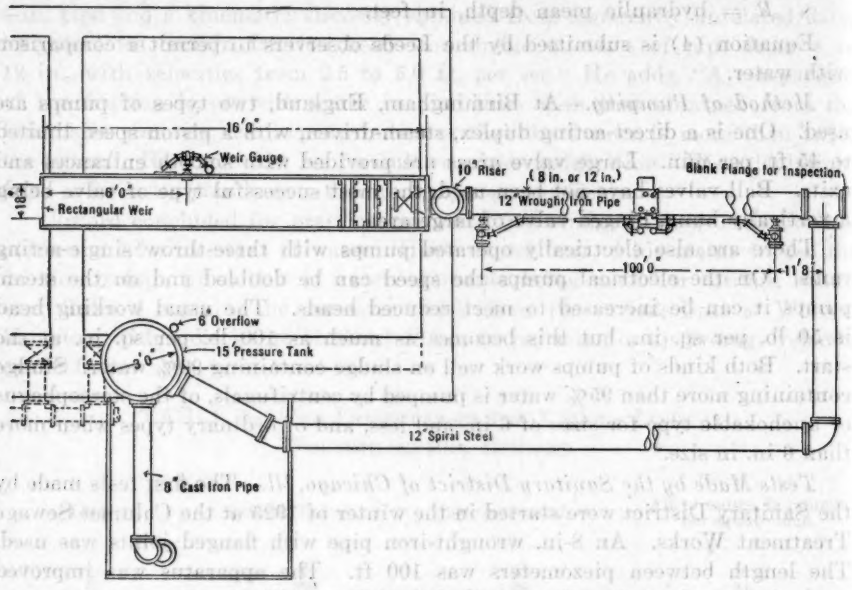

FIG. 6.—PLAN OF SYSTEM FOR LOSS-OF-HEAD TESTS.

In testing the 8-in. pipe, a 5-in. pump was used, the entire pipe loop being 8 in. in size. In testing the 12-in. pipe, an 8-in. pump was used, the entire pipe loop being 12 in. in size. The tests were operated at approximately constant heads.

In the arrangement and manipulation of the piezometers, considerable care and development were required to eliminate all the sources of error. Mercury gauges were soon abandoned for water gauges; ¾-in. taps for the piezometers were gradually enlarged to 2 in. A cleaning device, or ramrod, was developed to keep the taps clear. The piezometers were first connected on top, but were soon put on the side, because of difficulties with gas.

Various tests with the openings from ¾ in. to 2 in. showed difficulties in clogging and consequent inability to check measurements. A final arrangement was developed with a 2-in. tap from the test line, connected to a 3-in. tee, which served as a pressure well, and protected the ¾-in. line to the manometer. All the piezometer lines were filled with fresh water at the beginning of each test.

During the runs readings were taken simultaneously on the weir and on the manometers. The tests were run for increasing and for decreasing heads on the weir. Readings were taken at 30-sec. intervals for 2 to 3 min. for each successive head on the weir, depending on the fluctuation of the gauge.

The original piezometers were connected to mercury gauges, but these did not show sufficient differences of head to give accurate results. A manometer

gauge was erected in the middle of the test section, connected to the piezometers by ¾-in. lines, and supplied by a water pipe. At first, ¼-in. open manometers were used; then they were closed at the top and a common air pressure applied to bring the columns to a point easily read. The final type of piezometer used is shown (Fig. 7).

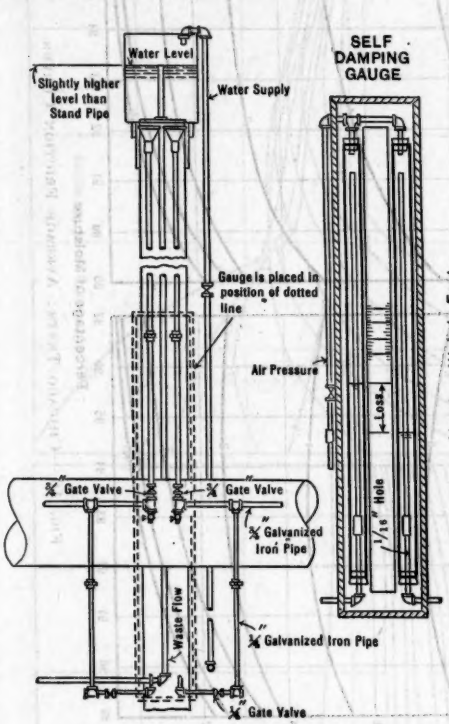

FIG. 7.—DETAIL OF PIEZOMETER.

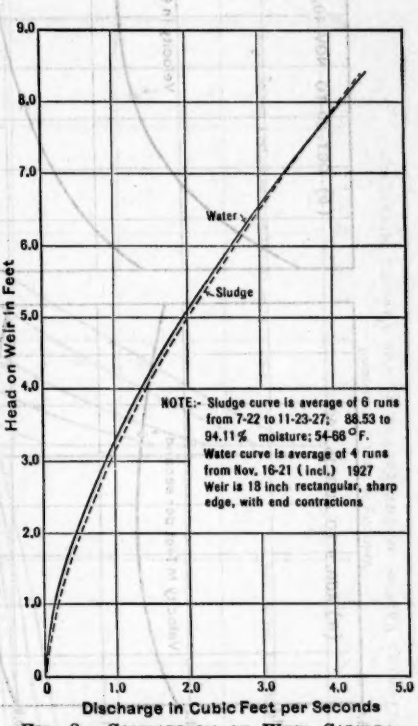
Head on Weir in Feet
Discharge in Cubic Feet per Seconds
Water
Sludge
NOTE:- Sludge curve is average of 6 runs from 7-22 to 11-23-27; 88.53 to 94.11% moisture; 54-66°F.
Water curve is average of 4 runs from Nov. 16-21 (incl.) 1927
Weir is 18 inch rectangular, sharp edge, with end contractions

FIG. 8.—COMPARISON OF WEIR CALIBRATION CURVES FOR SLUDGE AND WATER.

It was difficult to read the differences of head on the manometers because of fluctuations. The use of the self-damping gate patterned after that of Professor Ellis* proved satisfactory. The two manometer tubes (small inner tubes) have small holes at the bottom which slowly let water into the larger tube, thus dampening the water fluctuations. The levels in the large outer tubes were read. The pressure of the air over the top keeps the columns within range for easy reading.

The weirs were calibrated by a series of tests on sludges of various moistures by using the concrete tank to obtain volumetric measurements. In Fig. 8 an 18-in. rectangular, sharp-edged weir with end contractions was used. The sludge curve is the average of six runs made during the period from July 22 to November 23, 1927. The moisture content was from 88.53 to 94.11% at temperatures ranging from 54 to 66° Fahr. The water curve is the average of four runs from November 16 to November 21, 1927, inclusive. Discharges

* *Engineering News-Record*, September 22, 1927, Vol. 99, p. 478.

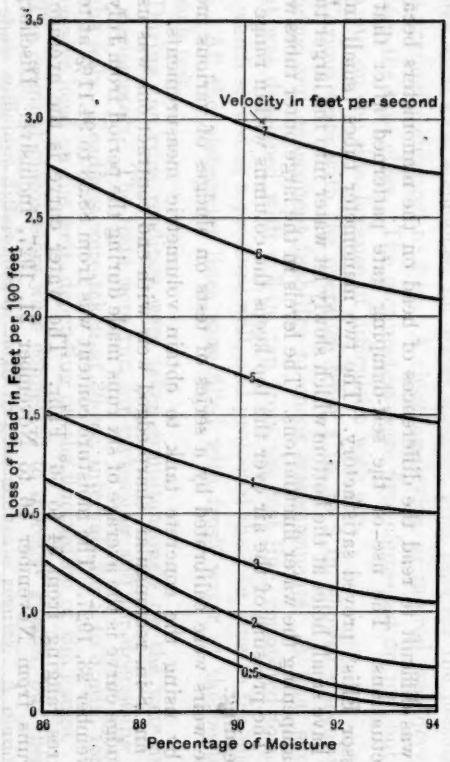

FIG. 9.—AVERAGE FRICTION CURVES.

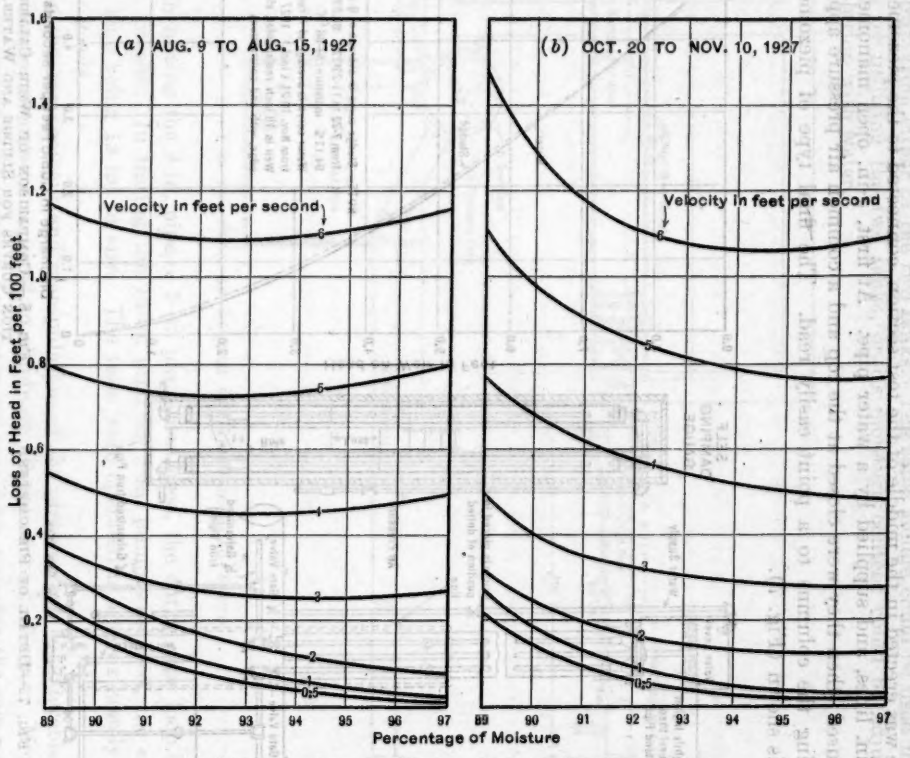

FIG. 10.—CHICAGO TESTS: AVERAGE FRICTION CURVES.

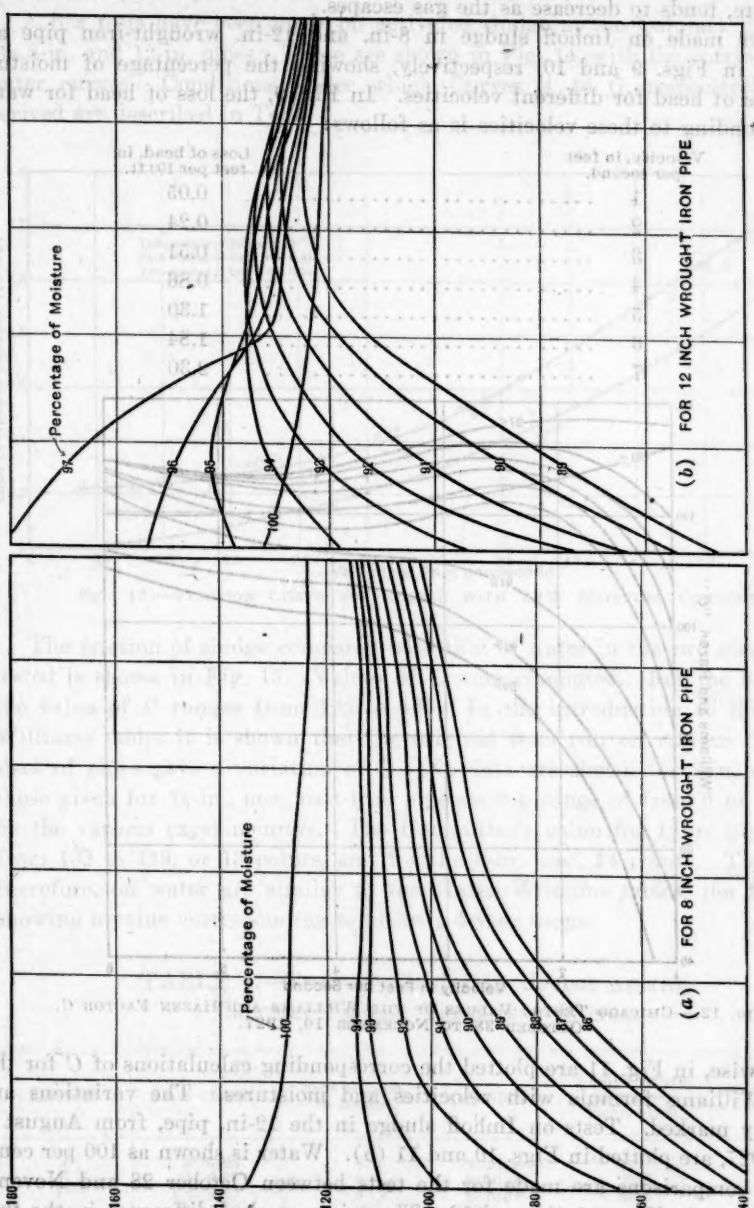

FIG. 11.—CHICAGO TESTS: VALUES OF THE FACTOR, *C*, FOR VARIOUS MOISTURES.

were found to be from 2 to 10% greater for sludge than for water. There is considerable gas in well-digested sludge from an Imhoff tank. The volume, therefore, tends to decrease as the gas escapes.

Tests made on Imhoff sludge in 8-in. and 12-in. wrought-iron pipe are plotted in Figs. 9 and 10, respectively, showing the percentage of moisture and loss of head for different velocities. In Fig. 9, the loss of head for water corresponding to these velocities is as follows:

Velocity, in feet per second.	Loss of head, in feet per 100 ft.
1	0.05
2	0.24
3	0.51
4	0.86
5	1.30
6	1.84
7	2.30

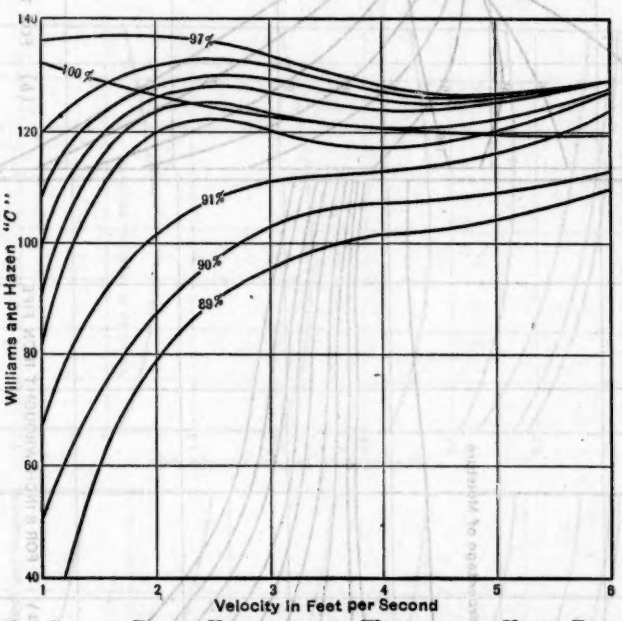

FIG. 12.—CHICAGO TESTS: VALUES OF THE WILLIAMS AND HAZEN FACTOR *C*, OCTOBER 28 TO NOVEMBER 10, 1927.

Likewise, in Fig. 11 are plotted the corresponding calculations of *C* for the Hazen-Williams formula with velocities and moistures. The variations are decidedly marked. Tests on Imhoff sludge in the 12-in. pipe, from August 9 to 15, 1927, are plotted in Figs. 10 and 11 (*b*). Water is shown as 100 per cent. Similar comparisons are made for the tests between October 28 and November 10, 1927, in Figs. 10 (*b*) and 12. There is a marked difference in the two sets of tests, each new batch of sludge giving somewhat different results quantitatively. In the tests on the 12-in. line, the loss of head at higher velocities is relatively less than for water for most of the moistures studied.

For comparison a typical run was selected, (Fig. 13) for which are plotted the individual test points as well as the water curve.

A few tests have been made on activated sludge at the Calumet Plant, on the 8-in. and 12-in. pipes. These are shown in Fig. 14, with the corresponding water curves. Conditions under which Curves *A* to *G*, respectively, were derived are described in Table 4.

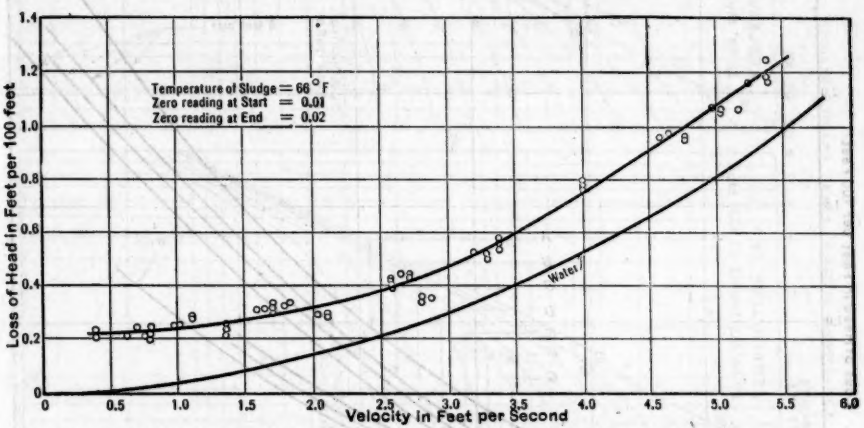

FIG. 13.—FRICTION CURVE FOR SLUDGE WITH 89% MOISTURE CONTENT.

The friction of sludge compared with that of water in the two sizes of pipe tested is shown in Fig. 15. Values of *C* were computed. For the 8-in. pipe, the value of *C* ranges from 122 to 136. In the introduction to the Hazen-Williams tables it is shown that the original tests run on various types and sizes of pipes gave a variation of *C*. No data are shown for 8-in. sizes, but those given for 12-in., new, cast-iron pipe show a range of from 9 to 13 points by the various experimentors. The Committee's value for 12-in. pipe ranges from 132 to 119, or 13 points, and for the 8-in. pipe, 14 points. The results, therefore, on water are similar to the Hazen-Williams tables, the factor, *C*, showing a value corresponding to about a 5-year usage.

TABLE 4.—TESTS ON ACTIVATED SLUDGE AT THE CALUMET SEWAGE TREATMENT WORKS.

Curve.*	Date of test.	Size of pipe, in inches.	Description.	Temperature, in degrees Fahrenheit.	Percentage of moisture.
A	9-16-26	8	Sludge	72	96.50
B	3- 8-27	8	Sludge	42.5	98.00
C	3- 9-27	8	Sludge	42	98.64
D	4- 1-27	8	Water	....	100.00
E	3- 8-27	12	Sludge	42.5	98.00
F	3- 9-27	12	Water	42	98.64
G	3-11-27	12	.....	....	100.00

* See Fig. 15.

Tests Made for Committee

To obtain data on the flow of sludge under different conditions, tests were made, from time to time, at Baltimore, Md., Chicago, Ill., and Syracuse, N. Y., in an effort to obtain data from which conclusions could be drawn. The material tested differs radically in each place.

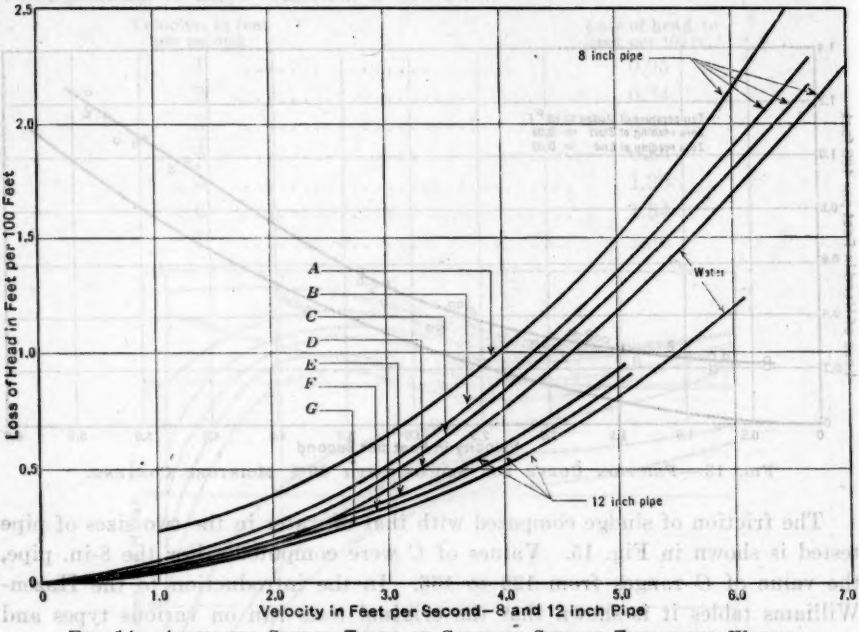

Fig. 14.—Activated Sludge Tests at Calumet Sewage Treatment Works.

At Syracuse, a fresh sludge from combined sewage was handled, derived from about 1-hour settling. The sewage passes through a grit chamber, 40 ft. long, in which a velocity of 1 ft. per sec. is maintained, then through a mechanically raked screen of ½-in. clear openings, before being pumped to the settling tanks.

At Baltimore, a sludge derived from 3-hour settling of a separate sewage was used. It had been in the digestion tanks from 2 to 12 weeks.

At Chicago, a well-digested sludge from a combined sewage was used in the main tests. It was taken from Imhoff tanks, after 3-hour settling, and from 4 to 6 months' digestion. The sewage passes through a grit chamber, 60 ft. long, in which a velocity of 0.8 ft. per sec. is maintained, after passing through a hand-cleaned bar screen of 1-in. clear opening.

The conditions of the tests also varied. At Syracuse, during the 1927 tests, a 4-in. pipe was used, 263.5 ft. long; at Baltimore, a 12-in. pipe, 2 250 ft. long; and at Chicago, an 8-in. and a 12-in. pipe, 100 ft. long between piezometers.

Results.—The tests at Chicago were made by the Sanitary District (see Fig. 15). The curve drawn for water in each case is based on actual tests in

Loss of Head in Feet per 100 Feet
Velocity in Feet per Second
Percentage of Moisture
Water
(a) 8 IN. WROUGHT STEEL PIPE TEMPERATURE 36-74° F
(b) 12 IN. WROUGHT STEEL PIPE TEMPERATURE 64-68° F
(c) 12 IN. WROUGHT STEEL PIPE TEMPERATURE 54-67° F
(d) 12 IN. WROUGHT STEEL PIPE TEMPERATURE 54-68° F

FIG. 15.—AVERAGE FRICTION CURVES FOR CALUMET SEWAGE TREATMENT WORKS, USING IMHOFF SLUDGE.

the pipe. Some of the tests indicate a friction loss in sludge less than that for water (see Fig. 15 (*b*), Fig. 15 (*c*), and Fig. 15 (*d*)). The only explanation appears to be the effect of gas in the sludge. There is a possible lubrication of the pipe wall by the sludge and a sliding of the sludge as a unit, due to the slippery pipe and cohesive characteristic of the sludge. All these conditions would tend to decrease friction.

These low frictions were not in evidence with 8-in. pipe, however, and as they were consistently found in 12-in. pipe, it may be that the size of pipe, or the ratio of circumference to area, has some bearing on the problem.

The work on the 8-in. pipe at low temperatures between 35 and 45° Fahr. indicated somewhat greater friction losses than for sludge at more than 50° Fahr. Every batch of sludge shows somewhat different results.

Two sets of runs were plotted in Fig. 10. The individual points on which the average velocity curves were drawn, and from which the corresponding values of C were calculated, are not shown (see Figs. 11 and 12). Apparently, for 12-in. pipe at 97% moisture and upward, C averaged from 125 to 129 with a 6-ft. velocity, while for water, C was 120. Sludges of 94% showed somewhat less friction throughout. For 8-in. pipe, the value of C was somewhat less than that for water.

Comparison of Tests.—In the tests of the Sanitary District, C varied but little at a velocity of 6 ft. per sec. for the range of moistures tested. The older Baltimore results show that with moistures of 92 to 96% the curves join at about 8 ft. per sec.

Description of Sludge Pipe Lines at Syracuse, N. Y.—The pipe line at Syracuse, described previously, is genuine, extra heavy wrought iron. The lengths are formed by screw couplings, taper tapped, and screwed home to butt the ends. Special precautions were taken to make a smooth interior. Long bends were used, with radii of more than 10 ft. The line was tested to 400 lb. per sq. in. after completion.

Manholes with hatch-boxes are provided every 500 ft. The cover is made with a fillet so as to maintain the same cross-section of the pipe. There are 2-in. gate-valves with hose connections. A ⅜-in. service cock is provided for connecting pressure gauges to locate stoppages and for making tests of friction losses. Blow-offs are provided at three points for emergencies. Provision is made for increasing velocities in the line by placing two or more pumps in service. Ordinarily, one pump is in service, the pumps being used in rotation every 6 hours. Clarified sewage can be admitted to the line for flushing purposes.

Three stoppages have occurred, all of which have been located readily by attaching a pressure gauge at the hatch-box connections. The time required to locate these stoppages varied from 0.5 to 1.5 hours. An additional hour or two was required to clear the line and resume pumping. The first interruption was caused by the cap of a metal drum lodging in the throat of a small Venturi meter which had been installed in the line for experimental purposes. The other stoppages were caused by deposits of fine sand and silt due to neglect in cleaning the grit chamber.

The line was cleared by removing the top of the first hatch-box beyond the stoppage, rodding with sewer-cleaning tools for a short distance, then placing a deflecting baffle of sheet metal within the open box and starting the sludge pumps with clear water. Solid plugs of sludge 20 to 30 ft. in length were forced out and deflected from the hatch-box by the metal baffle. After replacing the hatch-box cover, the line was flushed out with clarified sewage. No stoppage has occurred during 1926 and 1927.

Tests Made by the Syracuse, N. Y., Intercepting Sewer Board.—The first tests were made in 1922 by Cohen at Toronto. These were never considered satisfactory because of the severe weather conditions. After the construction of the treatment works at Syracuse, some observations were made on the pipe line, into which the sludge is discharged.

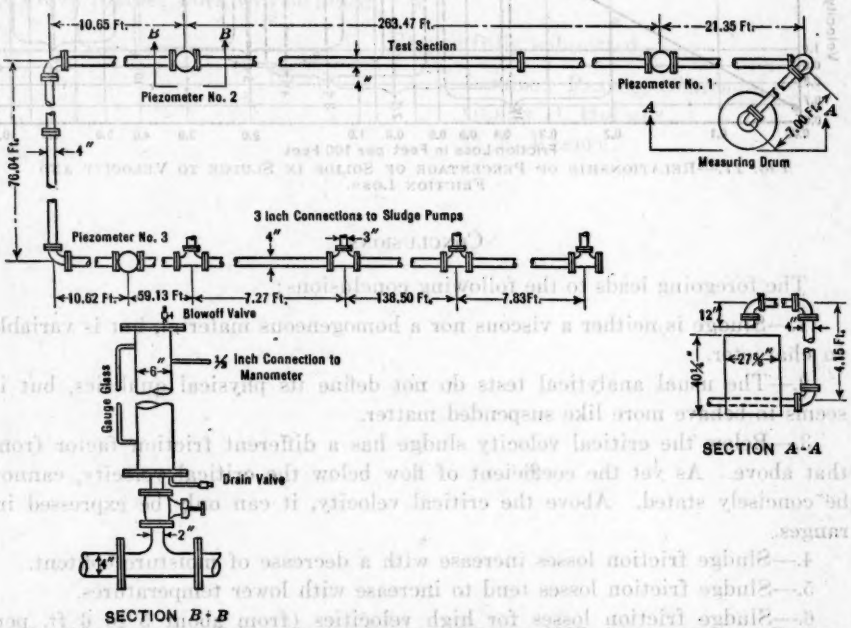

FIG. 16.—SYRACUSE TESTS: SLUDGE FRICTION STUDIES, 1927, ARRANGEMENT OF PIPING.

Finally, in 1927, a special layout was made for friction tests (Fig. 16). This comprised a 4-in. wrought-iron pipe, with flanged joints. The sludge was measured in a drum, the time to fill being taken with a stop-watch. The piezometers were of the water-column type, and the pipe was connected to the sludge pumps. In making the observations, pulsations of considerable magnitude were noticed, as well as a building up of the sludge friction and subsequent decrease. The head was varied by manipulating the speed of the pump. The results are summarized in Fig. 17.

The Syracuse tests show results in friction very much higher than those for Chicago. The points do not group as closely as the Chicago points, so that the results are indicated as zones (see Fig. 17). This means that on the tests, determinations were found to vary inside the lines indicated. The trend

of the data indicated that the loss of head might be approximately constant for a considerable range of velocity. These results resemble the curves obtained for slurry (Fig. 2).

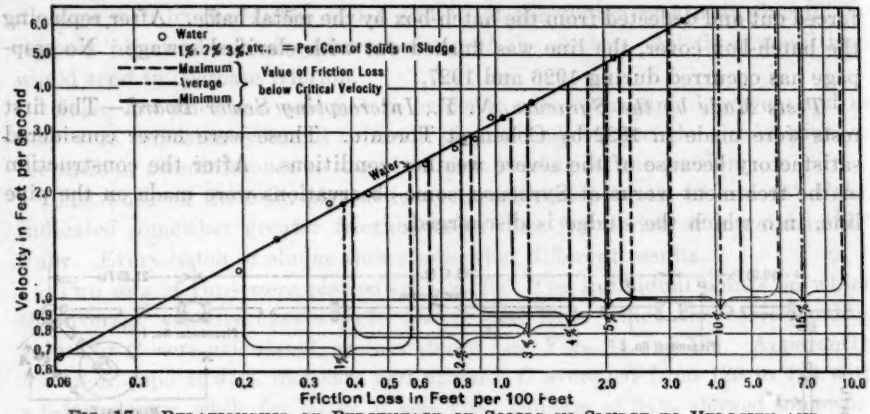

FIG. 17.—RELATIONSHIP OF PERCENTAGE OF SOLIDS IN SLUDGE TO VELOCITY AND FRICTION LOSS.

CONCLUSIONS

The foregoing leads to the following conclusions:

1.—Sludge is neither a viscous nor a homogeneous material, but is variable in character.

2.—The usual analytical tests do not define its physical qualities, but it seems to behave more like suspended matter.

3.—Below the critical velocity sludge has a different friction factor from that above. As yet the coefficient of flow below the critical velocity, cannot be concisely stated. Above the critical velocity, it can only be expressed in ranges.

4.—Sludge friction losses increase with a decrease of moisture content.

5.—Sludge friction losses tend to increase with lower temperatures.

6.—Sludge friction losses for high velocities (from about 5 to 6 ft. per sec., or more), tend to follow more nearly the characteristic laws for the flow of water.

7.—Friction losses for fresh or undigested sludge and for sludge from combined sewage are more erratic and the determination of a friction factor is correspondingly more difficult to obtain.

8.—Within the limits investigated no law of flow has been found.

RECOMMENDATIONS

It is recommended that:

(*a*) Further tests be made on as wide a range of pipe sizes as possible.

(*b*) The methods of test and of determining the quality of sludge be further investigated.

The Committee desires to acknowledge the assistance of the various engineers cited in the text and others who have courteously aided; in particular, the late Mr. Joseph Garfield, of Bradford, England, and to Mr. W. S. Tennant, of Glasgow, Scotland.

Further acknowledgment is due to Wilson G. Smith, Jun. Am. Soc. C. E., who has made most of the tests for the Sanitary District of Chicago, and has materially aided in compiling the data for the report, and to Mr. J. Stanley Gorsuch, who had charge of the field work at Baltimore, Md.

Further, the Committee desires to acknowledge the courtesy of the Bureau of Sewers at Baltimore, The Sanitary District of Chicago, and the Syracuse Intercepting Sewer Board in permitting the use of material contained herein.

The Committee hopes this effort will produce some constructive criticism by which further work will be aided.

Respectfully submitted,

LANGDON PEARSE, *Chairman,*
GLENN D. HOLMES,
C. E. KEEFER.

AMERICAN SOCIETY OF CIVIL ENGINEERS
INSTITUTED 1852

PAPERS AND DISCUSSIONS

This Society is not responsible for any statement made or opinion expressed in its publications.

FILTERING MATERIALS FOR WATER AND SEWAGE WORKS

PROGRESS REPORT OF THE COMMITTEE OF THE SANITARY ENGINEERING DIVISION*

The Committee on Filtering Materials for Water and Sewage Works has held two meetings during 1928, both in Cleveland, Ohio, one on April 21 and the second on November 9. Since the appointment of the Committee in 1925, five meetings have been held, and the interest of the members has been indicated by the full attendance at each meeting.

The effort of the Committee during the past year (1928) has been directed toward the development of laboratory methods to assist in the selection of filter materials for sewage trickling filters and the determination of the comparative suitability of materials considered for a given filter installation. The program of filter sand tests for water-works outlined in the progress report for 1927† has also been developed and put under way. A substantial appropriation for the work of the Committee has permitted the carrying forward of the comprehensive program of laboratory tests as recommended in the 1927 progress report, with reference to physical characteristics of material taken from a carefully selected number of existing sewage trickling filter installations.

Considerable interest has been exhibited in the activities of the Committee, both as regards the tests being made by the operation of small glass test filters for the size and depth of filter sand for water-works and in the various aspects of the studies relating to coarse-grained and fine-grained materials for sewage filters.

During the past years a number of sewage treatment experiments have been conducted, including the operation of test trickling filters, which have been used for varying lengths of time in connection with the study of different problems. A number of similar installations are now in operation or are

NOTE.—Written discussion on this paper will be closed in **January, 1930.**

* Presented at the meeting of the Sanitary Engineering Division, January 17, 1929.

† *Proceedings*, Am. Soc. C. E., April, 1928, Society Affairs, pp. 246 and 272.

contemplated. In the past much of the testing station work has been done with little reference to the physical characteristics of the filter media placed in the test filters. The Committee believes that a greater amount of useful information could be readily obtained from the operating results of such test filters as they relate to the character of the filter media. It is thought that the Committee may be in a position to make suggestions which would be helpful to a particular investigation under way, and at the same time aid in the development of operating data useful in the general study of trickling filter operation as it relates to filter media.

A program of experimental work is contemplated by the Committee, including the installation and operation under analytical study of small test filters at the sewage treatment works at Canton, Ohio, with various types of especially selected materials for filter media. The object of the tests is to study the question of filter efficiency as it is affected by the character of the filter media and the length of time the filters are in service. Similar experimental units are contemplated at two or more universities. The Committee will appreciate opportunities to co-operate in any such investigational program.

Testing and Selection of Sewage Trickling Filter Material

A comprehensive series of physical and chemical property tests has been conducted to determine the physical characteristics of a number of samples of material taken from sewage trickling filters which have been in service for some time. The samples were selected so that part of them represented material which had been in operation for a substantial period with apparent satisfactory service. Other samples were in part of material in which some evidences of deterioration have been found and, in part, of material that has not been in use for a sufficient length of time to determine the results of service. Under the first class of samples material was obtained from six filter installations where satisfactory service of ten years or more is indicated. Under the second classification samples were obtained from fourteen filter installations where some disintegration has taken place, or which have been in service less than ten years, or have been operated for only a part of the year.

These samples were subjected to a comprehensive analysis, including physical tests, chemical tests, and soundness tests by the sodium sulfate method. The laboratory work was performed by Professor E. A. Arnold, of the Case School of Applied Science, in Cleveland, under the immediate direction of George B. Gascoigne, M. Am. Soc. C. E., of the Committee. The results of these tests are given in detail in Appendix I, together with tentative conclusions relating to the value of the several tests.

The results of laboratory investigations must be compared with the operating results determined under actual service conditions before definite conclusions can be drawn relative to the value of the various tests. Accordingly, in connection with the laboratory investigations, comparatively careful and detailed study has been given to the trickling filter installations from which the test samples were obtained in an endeavor to determine data indicating what actually happens under service conditions. The experience of the Com-

mittee indicates that it is difficult to determine whether or not a filter medium in a given sewage trickling filter installation has deteriorated from disintegration or whether the deterioration of the filter bed is due to other causes.

However, it would appear that sufficient information has been obtained from the study of the operation of existing trickling filter installations during 1926, 1927, and 1928, supplemented by the results of the program of laboratory analyses which have been conducted, to substantiate the following tentative conclusions with reference to the testing and the selection of filter material for sewage trickling filters:

(*a*) Material for filter media should be sound, hard pieces with all three dimensions as nearly the same as possible, clean and free from dust, screenings, and other fine material. It should be of as nearly uniform size as possible, and the material should not disintegrate under service conditions either by breaking into smaller pieces or by crumbling into fine material.

(*b*) In selecting the most suitable material for a given installation from the available sources of material, careful investigation of the several sources of material should be made, including geological inspections of quarries, a study of the facilities for obtaining, screening, and cleaning the desired size of material, and a laboratory analysis of samples selected from the several sources to determine the comparative merits of the several materials as to the probability of the breaking down of the materials under filter service conditions. For this laboratory analysis samples should be selected from the various ledges in the several quarries or from the several possible storage piles.

(*c*) The investigation of the Committee to date indicates that the sodium sulfate soundness test is the most indicative test for predetermining the disintegrating qualities of the proposed material. Other physical tests, including the determination of specific gravity, the absorption, and porosity, give useful information. Chemical analyses are of value in determining the effect of impurities, such as earthy materials, iron, and sulfur, and, in some cases, lime and magnesium. Micro-structural analyses made by properly qualified examiners may also be useful.

(*d*) Great care should be taken in obtaining the filter material from a selected source to insure a uniform grade and size, free from dirt and fine matter. There is evidence to indicate that improper screening for size and cleaning of filter material for many of the existing trickling filter installations has been of influence in the deterioration of the filter beds, perhaps more so than the matter of breaking down or disintegration of the individual pieces of filter materials. It appears that proper sizing and cleaning of filter material cannot be obtained without special arrangements for screening and rescreening and perhaps washing of the material either at the source or at the filter plant immediately before placing the material in the filter bed.

The evidence at present available leads the Committee to conclude that the sodium sulfate soundness test is the most indicative of the available tests with reference to predetermining the probable disintegration of a proposed material for use as filter media in a trickling filter. This test involves certain details in laboratory technique which affect the results of the test. In order to reach a conclusion as to the practicability of the suggested procedure for making this test as given in Appendix III for the 1927 report*, when the test is made in different first-class standard materials testing laboratories, the Committee has arranged a program of co-operating laboratory tests by thirteen

* *Proceedings*, Am. Soc. C. E., April, 1928, Society Affairs, p. 267.

materials testing laboratories on a selected sample of limestone and a selected sample of slag.

In each case the sample contained ten pieces of materials with varying qualities as regards disintegration. Each set of samples sent to the several co-operating laboratories contained material as nearly identical as possible. The limestone pieces of the various grades were broken from the same block of stone and the slag pieces taken from the same run of slag. The sets of samples were sent to each laboratory in duplicate. The detailed results of this series of soundness tests are not yet available. A list of the co-operating laboratories, together with some data relating principally to the laboratory technique, are given in Appendix II. It is the hope of the Committee that this investigation will serve to evaluate the sodium soundness test as regards its utility for testing coarse-grained filter materials.

Fine-Grained Filter Material for Sewage Works

Investigation of fine-grained filter materials during 1928 has been somewhat retarded in favor of the comprehensive study of tricking filter materials. Some additional information has been collected in the way of further correspondence with State Sanitary Engineers and interested practicing engineers. However, this information has not been sufficiently extended to warrant any definite conclusions at the present time.

The information on fine-grained filter material for sewage work rather indicates that the use of sewage sand filters is confined to a few States and that at present very few new installations are being constructed. Considerable interest has been shown in the question of proper material for use in sludge beds. With the increased number of relatively large sewage treatment works being constructed, this phase of the study of fine-grained materials appears to be becoming relatively important and should warrant some definite study. Further discussion of fine-grained filter materials for sewage work appears in Appendix III.

Filter Materials for Water-Works

The program of study with reference to depth and size of sand for water filters by means of small glass test filters has been developed along the lines indicated in the progress report for 1927. The interest and enthusiasm with which this phase of the work has been undertaken by the men in charge of large filtration projects, is particularly gratifying to the Committee.

During the year the battery of small glass test filters, as proposed in the 1927 report, has been constructed by the water filtration departments of fourteen cities. All the units were identical as to character, size, and depths of sand used. To insure that the identical character of sand would be used in all the test filter units, a quantity of sand was obtained by James W. Armstrong, M. Am. Soc. C. E., of the Committee, screened to the several sizes required, and shipped to the several co-operating cities.

The work of constructing the test filters, supplying them with sand, and developing the details of the test procedure has taken up the greater part of

the year. No operating records of length are yet available. A number of minor difficulties relating principally to the routine of conducting the filter experiments have arisen. These are being "ironed out", and it is expected that these series of tests in the several co-operating cities will all be actively under way during the next several months. Present indications are that the tests should be extended over a period of perhaps two years before definite conclusions can be obtained from the results of operation.

A list of the co-operating cities, with the names of the engineers in charge of the work, is given in Appendix IV, together with some further data relative to the present status of the investigation. It is suggested that members of the Society particularly interested in filter materials for water-works would find a visit to these various places quite worth while. The Committee will appreciate comments that any member or other interested person cares to make relating to this and other phases of its investigational work.

Appendices

Data in more detail have been included in the form of Appendices to this report covering various phases of the current investigations of the Committee. These Appendices are as follows:

I.—Results of Investigational Work on Samples of Materials from Existing Sewage Trickling Filters.
II.—Co-operative Study of the Sodium Sulfate Soundness Test for Sewage Filter Materials by Thirteen Materials Testing Laboratories.
III.—Discussion of Fine-Grained Filter Material for Sewage Works.
IV.—Experiments Covering Depth and Size of Sand for Water Filters.

Recommendations

The investigation relating to the testing of coarse-grained filter material, the study of fine-grained materials, and the comprehensive investigation of filter sand for water-works are all incomplete. Also, there are other phases in the study of filter materials, for both sewage and water-works, which warrant consideration. Accordingly, it is respectfully recommended that the work of the Committee be continued.

The experience of the Committee has been that at least two meetings each year should be held to permit most progress and co-ordination of the work. There will be some minor expenses incident to the several lines of inquiry so that it is recommended that sufficient funds be provided to enable the Committee to continue the work as outlined.

Respectfully submitted,

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W. H. Dittoe,
George B. Gascoigne,
N. T. Veatch, Jr.

January 14, 1929.

APPENDIX I

RESULTS OF INVESTIGATIONAL WORK ON SAMPLES OF MATERIALS FROM EXISTING SEWAGE TRICKLING FILTERS

Introductory Statement

Early in the studies of the Committee, it was concluded that existing filter installations offered the best sources of data. Accordingly, an extended census was made to locate existing plants and to determine general data for each plant (see 1926 report*). Later, consideration was given to possible tests and methods for determining the physical characteristics of filter materials (see 1927 report†).

In order to correlate the laboratory tests with the effects, on the filter material, of operation of filter beds under service conditions, it was considered desirable to select samples from a number of typical trickling filter installations and apply the several laboratory tests.

During 1928, a comprehensive program of laboratory tests has been developed, including five physical property tests and a chemical analysis of samples of materials taken from twenty trickling filter installations.

The samples and operating data have been collected by various members of the Committee. The general program of the investigation has been carefully reviewed and approved.

The investigational work has been under the immediate direction of George B. Gascoigne, M. Am. Soc. C. E. The mass of detail work has been handled by Mr. F. W. Jones, of Mr. Gascoigne's office. The laboratory tests and chemical analyses have been made by Professor E. A. Arnold, of the Case School of Applied Science, of Cleveland, Ohio. Professor Arnold was employed by the Committee and made most of the laboratory tests and analyses during the summer in the school laboratories.

The Committee appreciates the effort put into this investigation by Mr. Gascoigne and his assistants. The discussion which follows, together with the presentation of the results of the tests, has been written largely by these men.

General Statement

During 1928 the investigational work of the Committee has progressed substantially and prior to discussing the information that has been obtained, it seems appropriate to outline some salient facts regarding the problem. At present, there appear to be three major considerations which may seriously affect the suitability of a trickling filter material. If some laboratory test or tests could be made available which would properly predict and evaluate its behavior in actual service, in respect to these considerations, some progress could be made toward establishing an intelligent basis for comparing the merits of any number of proposed materials. These essential considerations

* *Proceedings*, Am. Soc. C. E., April, 1927, Society Affairs, p. 164.
† *Loc. cit.*, April, 1928, Society Affairs, p. 243.

in determining the suitability of a trickling filter material may be classified as follows:

(*a*) Disintegration.
(*b*) Cementation and Consolidation.
(*c*) Ability to Unload.

The investigational work was primarily concerned with matters relating to disintegration.

Disintegration

Since the size of the filtering medium largely affects the capacity and efficiency of a trickling filter, the selected size should be maintained throughout the effective life of the filter, although there might be some minor breaking down in the size without affecting materially the efficiency of the filter. Thus, disintegration or the breaking down of the individual pieces of the material should be a minimum. Disintegration as applied to trickling filter material is here taken to mean the breaking down of the individual pieces to such sizes and in such a manner that the normal functioning of the bed will be handicapped and impeded. While absolute permanence is desirable, it is impractical to obtain this in all cases. Consequently, the extent of probable disintegration should be taken into account when considering proposed filtering materials.

When exposed to the atmosphere, filtering materials break down eventually due to natural weathering through the atmospheric agents of rain, frost, changes of temperature, and wind. The exposed surfaces of a trickling filter medium are subjected not only to natural weathering, but also to frequent alternate wettings and dryings. The wetting agent (sewage) contains appreciable quantities of dissolved carbon dioxide, also during cold weather the action of frost is greatly magnified due to frequent alternate freezing and thawings occasioned by the intermittent spraying of relatively warm sewage over the surface of the filter. When the temperature of the atmosphere is below freezing and the period between spraying is ½ hour, or more, the wetted surface will freeze between doses. Thus, the material in a trickling filter will be subjected to all the effects of natural weathering in greatly accelerated form, particularly as regards the mechanical effects of hydration and frost action. Furthermore, any chemical changes, such as the oxidation of iron or the slacking of free lime, are certain to be noticeable in much less time than under natural conditions.

Types of Disintegration.—Observations and the results of tests indicate that disintegration may occur to some extent in any stone or slag. The break-down may occur in several ways. First, there may a "splitting" of the material along seams. This may divide a particle into two pieces and end there, or again, there may be sufficient seams to permit of a break-down into many pieces. Splitting along seams appears to be more prevalent in hard dolomitic limestones than in the softer calcitic varieties. The spalling or chipping off of small pieces which sometimes occurs, may also be due to seams. Again, the disintegration may take the form of "blowing up". In such cases the

hard material suddenly gives way and is reduced to many small pieces. This type of disintegration is found in certain kinds of cherty limestones. The third type of disintegration, and one which seems to be mostly confined to the softer varieties of limestones and to the slags, is the gradual "crumbling" away of the particles. Such disintegration, while eventually even more disastrous than the other types, is not so noticeable to the casual observer. However, as the particles crumble away to granular bits, the possibility of consolidation and clogging becomes more and more likely.

Where hard material disintegrates or splits into small pieces which still retain the characteristics of the parent piece, the ill effects do not seem to be so severe as when a complete breakdown to fines or dust occurs. In this connection, it is readily seen that where smaller pieces are made from larger, with some of the large pieces unaffected, the voids are materially reduced, but the surface area is increased. Consequently, if the dosing is not too high, so as to drown out the filter, it is possible that the accomplishments of the filter will not be greatly affected. On the other hand, if the dosing is higher than the reduced voids can handle, pooling and internal putrefaction are almost certain to occur.

It may be concluded, therefore, that any material in which the individual particles show a tendency to disintegrate other than by simple subdivision into two pieces, should be viewed with suspicion. It is only as the original sizing of the filtering material is maintained that the effectiveness of the filter can be assured. Consequently, it would appear that the quantity of undersized material due to disintegration from natural or artificial causes should be definitely limited.

Testing for Disintegration.—Tests which assimilate the conditions of operation under service would appear to be the best for predetermining the disintegrating qualities of proposed filter materials. Actual freezing and thawing tests would seem to be the most desirable. Such tests require a large number of cycles of alternate freezings and thawings to compare with service conditions and hence they are rather expensive and time-consuming. Various accelerated tests have been proposed of which the sodium sulfate soundness test appears the most likely.

Various tests have been developed largely in connection with road materials, which may give some indication of the physical characteristics of materials by means of which the resistance to disintegration may be predetermined. Since sewage has a relatively complex chemical composition, it would appear that a knowledge of the chemical composition of a filter material would be pertinent. The program of tests included all these considerations.

Cementation and Consolidation

On some experiences with the older trickling filter installations, especially where clinkers and other relatively friable materials have been used, it was found that these materials were often reduced to fines which were cemented together or compacted or consolidated so completely as to render the filter inoperative and non-effective. It is highly desirable, therefore, to use a material which will not reduce to fines or become consolidated.

The laboratory testing work undertaken to date by the Committee applies to the subject of consolidating only in so far as the tests indicate the possibility of a material breaking into excessive quantities of fine matter. More extended studies are desirable relating particularly to the various factors which cause consolidation.

Ability to Unload

The clogging of trickling filters may occur in the main from two causes: First, by a breaking down of the filtering material to fines which compact, as discussed previously; and, second, by the retention within the voids of sewage substances which should wash out. In the first instance, the likelihood of clogging will depend on the extent and manner in which disintegration has occurred. In the second instance, the extent of clogging will depend on the size of material (which also will be affected by disintegration), the amount of suspended matter applied, and possibly the relative roughness of the material.

Testing for Ability to Unload.—There seems to be no definite test to determine relative smoothness and roughness of filtering materials. The investigations of the Committee to date (1928) indicate that this phase of the study of filter materials is complicated and definite data can only be obtained from detailed observation of filters in operation, more particularly the operation of test filters where definite control can be had over the many variables. Consideration has been given to the installation of experimental trickling filter units for the study of the effect of roughness of material with respect to its unloading ability and the resultant effect on the over-all filter efficiency.

Program of 1928 Investigational Work

A comprehensive study, with reference to the proper selection of trickling filter materials for a given installation, must include consideration of several factors, such as probable disintegration effect of surface roughness on unloading, effect of variation in size of material, etc. Such a study would be rather extended and probably would require the operation of experimental trickling filter units. It has been thought advisable for the Committee to concentrate its efforts on the disintegration factor, on which the other factors are more or less contingent.

In 1927, certain tentative methods of sampling and analyzing filtering materials were proposed.* Furthermore, and from that time, the work has involved the collection of samples having known records of service in existing trickling filter plants. Some of the samples should be of materials in which practically no disintegration had taken place over a number of years, and others of materials in which some disintegration has occurred under similar service conditions. For the purpose of identification and easy comparison, the former were termed satisfactory materials and the latter partly satisfactory.

With these viewpoints in mind, the Committee canvassed the trickling filter installations of various sizes in different sections of the country, and

* *Proceedings*, Am. Soc. C. E., April, 1928, Society Affairs, p. 247.

decided that samples should be collected from each of the places given subsequently, in accordance with directions sent out by the Committee. Naturally, installations with about 10 to 15 years of continuous service were viewed with favor.

The list of installations from which samples were taken, where the filter material has proved satisfactory for 10 years or more of continuous use, is as follows:

Location.	Kind of Material.
Baltimore, Md.	Trap rock
Plainfield, N. J.	Trap rock
Rochester, N. Y.	Limestone
Gloversville, N. Y.	Limestone
Madison, Wis.	Dolomite
Fitchburg, Mass.	Trap rock and granite

Plants from which samples were taken, where the filter material is either new, or has been in service less than 10 years, or has been operated for only a part of the year, or where some disintegration has taken place, are:

Location.	Kind of Material.
Marion, Ohio	Limestone
Lincoln, Nebr,	Quartzite
Decatur, Ill.	Limestone
Schenectady, N. Y.	Limestone
Reading, Pa.	Slag
Pontiac, Mich.	Gravel
Polk, Pa.	Sandstone
Springfield, Mo.	Limestone
Mason City, Iowa	Limestone
Canton, Ohio	Slag
Fort Sheridan, Ill.	Limestone
Cleveland, Ohio (Southerly Plant)	Limestone
Fort Worth, Tex.	Limestone
Columbus, Ohio	Limestone

Directions for taking the samples were given to various members of the Committee who assumed the responsibility for the accuracy of the necessary data and the samples.

Instructions for Collecting Samples and Data Required.—An instruction sheet was prepared, including detailed directions covering the way the samples of material were to be taken from the filter bed as well as a form sheet for recording pertinent data. The form is as follows:

AMERICAN SOCIETY OF CIVIL ENGINEERS

SANITARY ENGINEERING DIVISION

Committee on Filtering Materials for Sewage and Water Works

Instructions for Collecting Sample and Data Required

Collection and Sample: A sample of material representative of that which has been used under normal operating conditions should be taken about midway between nozzles and at a location which, by observations, indicates typical use of the filter. It seems best to take this sample by digging down into the

filter about halfway and picking off the exposed section, from the top down, various stones. Samples for shipment *by freight* to this office (1149 Leader Building, Cleveland, Ohio) should consist of about a cubic foot and should be boxed.

It might be well also to collect from the surface of the filter pieces of material which have withstood conditions and some pieces which have obviously broken down. Where possible, all pieces of the disintegrated particle should be included. A small carton might be used for this. Also, some pieces should be taken from the edges or unused portion of the filter.

Collection of Data: The following information relating to the filter sampled should be obtained by the person taking the sample. Valuable information can often be obtained from the laborers as well as from those in responsible charge.

City and name of plant.....
When placed in operation.....
Designed by .....
Built by .....
Official in charge of operation.....
Local operator .....
Number of employees on plant.....
Technical supervision, and how long.....
Population designed for.....
Population served: At start of operation.....
At present .....
Sewage flow filtered in M. G. D.....
At start of operation.....
At present .....
Minimum rate, dry weather, M. G. D.....
Maximum rate, dry weather, M. G. D.....
Maximum rate, wet weather, M. G. D.....
Total area of filters.....
Type of dosing tank.....
Period of dosing.....
Period of rest.....
Are filters operated continuously winter and summer.....
Temperatures .....
Maximum air..... Maximum sewage.....
Minimum air..... Minimum sewage.....
What was size of material originally placed.....
Has disintegration occurred.....
Is disintegration noticeable on surface.....
Are there any indications that fine material has worked down into the bed .....
Is the period of unloading continuous or spasmodic.....
When does most unloading occur.....
Are there pools on the surface.....
Are the pools due to surface growths or fine compacted material.....
What is the appearance of the surface growths.....
What type of final settling tank is used.....
Is the sludge from those tanks light or heavy.....
Does it contain obvious washings from the filter material.....
Do the under-drains of the filter clog.....
Where was the source of the filtering material.....
Is it considered satisfactory.....

Samples Received.—In all, twenty trickling filter installations were selected for sampling. They represent periods of service from "just starting" to 20 years. Of the total, six which had been in service without apparent or appreciable disintegration for 10 years, or more, were considered to be satisfactory, and fourteen were either in the class of filters in service less than 10 years, or filters in which obvious disintegration had occurred. Owing to difficulties in making contacts with some of the operators, it was impossible to get all the samples in time to make the necessary tests during the summer of 1928. However, samples were received from sixteen plants in time to be included in the tests.

The places not included in the tests are Fort Worth, Tex.; Polk, Pa.; Madison, Wis.; and Canton, Ohio, from all of which samples are now (January, 1929) on hand.

Testing Facilities.—In order to give proper attention to technique, it was deemed advisable to secure the services of some one fully competent to make chemical and physical analyses, accordingly it was decided to employ Professor E. A. Arnold, of Case School of Applied Science. This permitted Professor Arnold to make the analyses in his own laboratory where adequate space and facilities were available. At the same time Mr. Gascoigne was able to keep in close touch with the work as it progressed.

Preparation of Samples.—Most of the samples, except one or two which were collected by automobile, came by freight. The sample, on delivery at Case School, was opened and examined. Where possible, it was so prepared that good stone was separated from that which had undergone disintegration. These were carefully examined and samples were taken from each for both chemical and physical tests. In securing the sample for chemical analyses, chips were taken from various pieces representing the kinds of material. This was ground up to a fine powder in a ball mill, and from this fine powder, after thoroughly mixing, a sufficient quantity was taken to conduct the analyses. For the sodium sulfate test a composite consisting of both good material and where possible also, that which had undergone disintegration, was selected. For the density and porosity tests, sufficient samples were taken to represent all types of material present.

Determinations Made and Methods Used.—The analyses may be divided into two divisions, namely, the physical tests and chemical tests. Under the physical tests may be listed the following: Apparent specific gravity; absorption; porosity; soundness by sodium sulfate; and abrasion.

Under the chemical tests, the following determinations were made: Insoluble matter; iron oxide; aluminum oxide; calcium; magnesium; and sulfur.

In making the physical tests the methods outlined in the progress report of 1927* were followed, except for the specific gravity. In this test, instead of making the determination on a volume of stone and obtaining the average of the whole, the tests were made on individual pieces, which represented varieties of the material noted in the sample. In this manner many possible differences in the specific gravity of the different types could be better detected. The absorption and porosity figures were calculated from the same samples on

* *Proceedings*, Am. Soc. C. E., April, 1928, Society Affairs, pp. 268-270.

which the specific gravity was taken. Each piece analyzed weighed approximately 50 grammes.

In making the abrasion test it was found that the method called for in the progress report of 1927 could not always be followed in detail as to the number of pieces to be used, on account of the fact that in some of the samples the pieces were rather small. Consequently, where the pieces were small a 5-kg. sample was used, irrespective of the number of pieces.

Chemical Analyses.—The chemical analyses were run in accord with the methods suggested in the 1927 report,* except for the sulfur, which was determined according to a method given in "Inorganic Quantitative Analyses" by Scott, under the title of "The Determination of Total Sulphur in Pyrites". Sulfur in the slags which is present as sulfide sulfur, was determined by a modification of the evolution method for sulfur in steel.

In the limestones, the iron was separated from the alumina and was further separated into FeO and Fe_2O_3. Calcium and magnesium were reported as carbonates or as oxides in the limestones and slags, respectively. Sulfur was reported as S in the case of limestones, and as CaS in the case of slags.

In determining the insoluble matter, it is evident that some of it is silica and part is silicate which is insoluble in hydrochloric acid. Accordingly, where the amount of insoluble matter was considerable, a test was made to determine what percentage was volatile with hydrofluoric acid, representing true silica; and what part remained unaffected by hydrofluoric acid. This would be clay and other earthy material.

Results of Tests

The program of laboratory tests has been applied to samples of filter material from sixteen trickling filter installations. A brief description of each of these is given herewith. In Table 1 is listed the absorption, specific gravity, and porosity, and Table 2 includes the chemical analyses.

Baltimore, Md.

The City of Baltimore, Md., has a total of 30 acres of trickling filters divided into ten beds of 3 acres each. They are 8½ ft. deep and are composed of trap rock and granite obtained from Havre de Grace, Md. They were first placed in operation in November, 1911, and have been in continuous service, throughout the year, ever since. They were designed for a population of 1 000 000 persons, and now serve 750 000, with an average flow of 60 000 000 gal. daily. The size of material is 1 to 2½ in.

Officials in charge of the plant state that very little, if any, disintegration has occurred, and not any is noticeable on the surface. Small quantities of very fine material have been observed in the final settling tanks from time to time. There has been very little surface clogging although there is some reddish brown growth. The final tank is of the plain settling type and the filters are said to unload spasmodically each year mostly in the spring and autumn. The sludge in the final tank is very light. The filtering material is considered to be entirely satisfactory.

* *Proceedings*, Am. Soc. C. E., April, 1928, Society Affairs, p. 270.

TABLE 1.—ABSORPTION (48 HOURS). SPECIFIC GRAVITY AND POROSITY OF FILTERING MATERIALS, ARRANGED IN ASCENDING ORDER OF AVERAGE ABSORPTION.

Item No.	Place.	Material.	Number of pieces.	Absorption (Percentage of Total Weight).			Specific Gravity.			Porosity (Percentage of Total by Weight).			Remarks.
				Maximum.	Minimum.	Average.	Maximum.	Minimum.	Average.	Maximum.	Minimum.	Average.	
1	Lincoln, Nebr....	Quartzite..	9	0.50	0.08	0.16	2.72	2.64	2.66	1.30	0.18	0.43	Sound stone.
2	Rochester, N. Y...	Limestone.	4	0.48	0.06	0.21	2.71	2.62	2.68	1.28	0.16	0.55	Unused stone.
3	Baltimore, Md.....	Trap-rock..	9	0.76	0.02	0.27	3.07	2.76	2.92	2.11	0.13	0.81	Sound stone.
4	Baltimore, Md.....	Granite....	3	0.31	0.26	0.28	2.69	2.68	2.69	0.83	0.69	0.73	Sound stone.
5	Rochester, N. Y...	Limestone.	3	0.47	0.18	0.31	2.70	2.62	2.65	1.22	0.50	0.81	Used stone, sound.
6	Schenectady, N. Y.	Limestone.	6	0.58	0.14	0.32	2.83	2.67	2.78	1.51	0.48	0.86	Sound stone.
7	Gloversville, N. Y.	Limestone.	5	0.68	0.15	0.33	2.83	2.72	2.79	1.83	0.43	0.89	Sound stone.
8	Marion, Ohio.....	Limestone.	3	0.58	0.49	0.54	2.77	2.76	2.77	1.58	1.33	1.47	Sound stone, sub-surface.
9	Springfield, Mo....	Limestone.	5	0.90	0.35	0.70	2.68	2.60	2.64	2.29	0.92	1.76	Composite.
10	Plainfield, N. Y....	Trap-rock..	7	1.40	0.40	0.79	2.93	2.77	2.84	3.83	1.15	2.20	Sound stone.
11	Schenectady, N. Y.	Limestone.	2	0.86	0.85	0.86	2.72	2.59	2.65	2.26	2.17	2.21	Disintegrated stone.
12	Cleveland Ohio...	Limestone.	5	1.34	0.32	0.91	2.71	2.55	2.61	3.31	0.85	2.50	Composite.
13	Mason City, Iowa.	Limestone.	5	1.35	0.48	0.99	2.68	2.60	2.63	3.40	1.14	2.52	Sound stone.
14	Marion, Ohio	Limestone.	9	1.56	0.75	1.16	2.69	2.63	2.65	3.95	1.97	2.99	Disintegrated stone.
15	Columbus, Ohio...	Limestone.	6	1.66	0.54	1.17	2.74	2.61	2.67	4.16	1.47	3.02	Sound stone.
16	Mason City, Iowa.	Limestone.	10	2.43	0.39	1.34	2.73	2.58	2.64	5.95	1.03	3.37	Disintegrated stone.
17	Marion, Ohio.....	Limestone.	3	1.60	1.08	1.39	2.70	2.66	2.68	4.08	2.84	3.60	Sound stone, surface.
18	Reading, Pa.....	Slag.....	5	1.33	0.31	1.46	2.76	1.70	2.40	16.04	0.79	5.24	Bed No. 1, large pieces.
19	Fort Sheridan, Ill.	Limestone.	6	3.99	0.83	1.63	2.74	2.36	2.60	8.61	2.32	3.89	Composite.
20	Decatur, Ill.....	Limestone.	10	2.89	0.59	1.72	2.81	2.66	2.70	5.53	1.63	4.08	Small pieces.
21	Decatur, Ill.....	Limestone.	10	2.66	1.18	1.82	2.73	2.62	2.67	6.52	2.49	4.48	Large pieces.
22	Columbus, Ohio...	Limestone.	3	2.13	1.80	1.91	2.62	2.60	2.61	5.32	4.49	4.58	Partly disintegrated stone.
23	Columbus, Ohio...	Limestone.	...	....	....	....	....	....	....	....	....	....	Composite sample of disintegrated stone.
24	Pontiac, Mich.....	Sandstone.	4	5.91	2.80	2.70	2.73	2.29	2.54	11.91	8.43	4.59	Disintegrated, most of Pontiac material hard igneous gravel.
25	Columbus, Ohio...	Limestone.	6	4.28	1.89	2.72	2.56	2.37	2.49	9.29	4.57	2.49	Sample of partly disintegrated stone from all types of material.
26	Columbus, Ohio...	Limestone.	3	2.92	2.81	2.86	2.47	2.45	2.46	6.71	6.49	6.28	Considerably disintegrated stone.
27	Reading, Pa.....	Slag.....	4	5.80	1.88	3.88	2.47	1.81	2.06	9.51	4.46	7.12	Bed No. 4.
28	Pontiac, Mich.....	Sandstone.	3	6.79	4.92	6.15	2.29	2.00	2.17	13.41	8.94	11.80	Disintegrated stone (see note under Item No. 24).
29	Reading, Pa.....	Slag.....	6	8.84	1.58	6.35	2.61	1.53	1.97	12.49	3.97	10.42	Bed No. 3.
30	Reading, Pa.....	Slag.....	10	13.35	0.70	6.37	2.83	1.55	2.10	18.65	1.95	10.41	Bed No. 1, small pieces.
31	Pontiac, Mich.....	Shale.....	3	12.05	3.87	8.38	2.36	1.98	2.15	19.17	8.36	14.56	.....

TABLE 2.—CHEMICAL ANALYSES OF FILTER MATERIALS ARRANGED IN DESCENDING ORDER OF INSOLUBLE MATTER (SILICA, ETC.) CONTENT.

Item No.	Place.	Material.	Chemical Content (Percentages by Weight)						Remarks.
			Insoluble matter, silica, etc.	Alumina, Al_2O_3.	Iron oxide, FeO;[1] Fe_2O_3.[2]	Calcium oxide;[1] carbonate.[2]	Magnesium oxide;[1] carbonate.[2]	Sulfur, S;[1] CaS.[2]	
1	Lincoln, Nebr.	Quartzite	86.78	1.77	8.52[1]	Trace	None	Trace	Sandstone.
2	Columbus, Ohio.	Limestone	82.06†	0.75*		9.78[2]	6.63[2]	0.14[1]	*$Al_2O_3 + Fe_2O_3$. †0.61% non-volatile with HF; disintegrated stone.
3	Pontiac, Mich.	Sandstone	81.54	11.25	3.06[2]	10.06[1]	Trace[1]	Trace[1]	Red sandstone disintegrated.
4	Pontiac, Mich.	Sandstone	74.64	7.64	10.21[2]	1.62[1]	Trace[1]	Trace[1]	Gray sandstone disintegrated.
5	Baltimore, Md.	Granite	74.32	19.02*		2.77[1]	0.32[1]	None	Sound stone, *$Al_2O_3 + Fe_2O_3$.
6	Mason City, Iowa.	Calcareous sandstone	62.43†	2.17	1.57[2]	27.59[2]	4.33[2]	0.14[1]	Small sandstone piece crumbled during washing; †16.35% non-volatile with HF.
7	Baltimore, Md.	Trap	55.24	16.77	14.77[1]	5.65[1]	3.21[1]	0.67[1]	Sound stone.
8	Schenectady, N.Y.	Limestone	55.11†	1.80	1.76[2]	36.16[2]	4.85[2]	0.35[1]	Disintegrated stone, 22.12% non-volatile with HF.
9	Plainfield, N.J.	Trap	49.27	16.49	12.68[1]	10.37[1]	8.01[1]	0.16[1]	Sound stone.
10	Rochester, N.Y.	Flinty limestone	43.37†	0.11	4.62[2]	49.12[2]	2.45[2]	0.15[1]	Used stone; †1.42% non-volatile with HF.
11	Rochester, N.Y.	Flinty limestone	42.95	2.16	5.54[2]	46.43[2]	2.93[2]	0.11[1]	Unused stone.
12	Reading, Pa.	Slag	38.46	13.95	1.80[1]	35.29[1]	8.41[1]	1.75[2]	Bed No. 1, small pieces.
13	Reading, Pa.	Slag	37.73	13.04	0.49[1]	37.02[1]	9.88[1]	1.67[2]	Bed No. 4.
14	Reading, Pa.	Slag	37.35	12.75	0.70[1]	38.15[1]	8.03[1]	2.51[2]	Bed No. 1, large pieces.
15	Reading, Pa.	Slag	36.48	12.59	0.62[1]	35.42[1]	12.39[1]	1.27[2]	Bed No. 3.
16	Springfield, Mo.	Limestone	14.71†	0.18	2.12[2]	82.73[2]	0.16[2]	None	†0.44% non-volatile with HF; composite.
17	Schenectady, N.Y.	Limestone	14.30†	1.51	1.57[2]	52.82[2]	29.70[2]	0.12[1]	Sound sub-surface; †5.18% non-volatile with HF.
18	Decatur, Ill.	Limestone	7.40†	0.01	1.88[2]	50.00[2]	40.87[2]	None	Large pieces; 2.29% non-volatile with HF.
19	Decatur, Ill.	Limestone	7.35	0.37	1.57[2]	50.15[2]	40.59[2]	None	Small pieces.
20	Gloversville, N.Y.	Limestone	6.99	1.16*		55.24[2]	36.64[2]	Trace	Sound stone, *$Al_2O_3 + Fe_2O_3$.
21	Marion, Ohio	Limestone	6.80†	2.60	1.28[2]	63.84[2]	25.32[2]	None	Sound surface stone; †1.94% non-volatile with HF.
22	Cleveland, Ohio.	Limestone	6.32	0.47*		80.89[2]	11.95[2]	0.24[1]	Sound stone, *$Al_2O_3 + Fe_2O_3$.
23	Marion, Ohio.	Limestone	5.28	1.85	1.20[2]	74.25[2]	17.02[2]	0.41[1]	Sound sub-surface stone.
24	Fort Sheridan, Ill.	Limestone	3.54	0.79	1.19[2]	53.12[2]	41.15[2]	None	Composite.
25	Columbus, Ohio.	Limestone	3.36†	0.47*		63.82[2]	32.05[2]	Trace	Disintegrated stone; †0.74% non-volatile with HF; *$Al_2O_3 + Fe_2O_3$.
26	Marion, Ohio.	Limestone	3.26†	0.63*		83.65[2]	12.04[2]	0.13[1]	Disintegrated stone; †0.94% non-volatile with HF; *$Al_2O_3 + Fe_2O_3$.
27	Mason City, Iowa.	Limestone	0.17	0.55*		98.38[2]	Trace	Trace	Sound sub-surface stone; *$Al_2O_3 + Fe_2O_3$.

Results of Tests.—The sample of stone from the Baltimore filters consisted of a mixture of trap rock and granite and was received in a box, containing two small cartons. In one carton was a specimen of trap from the top of the bed, and in the other a specimen of unused trap rock. It was explained that no samples showing disintegration were included, as disintegration had not occurred. Consequently, the sample was taken from the surface of the bed.

Determinations of apparent specific gravity, absorption, porosity, and chemical analyses were made on separate samples of both the granite and trap rock. The abrasion test and sodium sulfate test were made on a composite sample including both granite and trap. Three pieces of granite were tested for apparent specific gravity, absorption, and porosity (see Table 1, Item 4). The chemical analyses of a composite sample made by grinding up chips from a large number of pieces of granite is shown in Table 2, Item 5. Tests on nine pieces of trap rock consisting of both used and unused material are shown in Table 1, Item 3. The chemical analysis of a composite of the trap rock is given in Table 2, Item 7.

In the sodium sulfate test ten pieces, three of granite and seven of trap rock, having a total weight of 2 029 grammes, were subjected to twenty cycles. After the twentieth cycle, the pieces were washed, dried, screened through a 1-in. sieve, and again weighed, the loss of material less than 1 in. in size amounting to 19 grammes, or 0.94% of the whole. One piece parted at the end of the ninth cycle along a pyrite plane.

Twenty-five pieces of trap rock and the same number of pieces of granite, or a total of 5 038 grammes, were used in the abrasion test. After the required 10 000 revolutions there were fifty-three pieces having a total weight of 4 920 grammes. This represents an abrasion loss of 118 grammes, or 2.34% wear. A test of the fines showed 93.6% passing a 100-mesh sieve.

Conclusion.—These tests show that the granite has an average specific gravity of 2.69 whereas the average specific gravity of the trap rock is 2.92 (Table 1). The absorption and porosity of the granite average 0.28 and 0.73, respectively, while the corresponding figures for the trap are 0.27 and 0.81. The chemical analyses of both the trap rock and granite are typical of igneous rocks in these classifications.

The sodium sulfate test indicates a satisfactory material after twenty cycles. The piece that parted along a pyrite seam after the ninth cycle was unused material. This cannot be considered a failure as no further disintegration was noted. The abrasion tests show a relatively low percentage of wear.

Decatur, Ill.

The 3 acres of trickling filters in the sewage plant at Decatur, Ill., have been in operation for probably 60% of the time. They were out of service for long periods because of floods and new construction work, since 1923. The filters consist of broken limestone, mostly 1 in. to 2 in. in size and 6½ ft. in depth. They were designed for a population of 60 000, and, in 1925, served 50 000. However, the tributary industrial sewage has a population equivalent much greater than the actual population served.

Some clogging occurred during the early years of operation. This is explained in part by the fact that the sewage contained abnormal quantities of industrial wastes, with a high oxygen demand, from which some suspended matter was carried over from the settling tanks. The clogging on the surface was relieved by means of picking and racking and this resulted in the breaking up of some of the surface stone which worked down into the bed. Consequently, at present, while some fine material may be found below the surface, the surface material is apparently whole. Within the last year (1928) modifications in the preliminary treatment of the sewage permit much higher rates on the filters than formerly, without surface clogging. Unloading occurs seasonally. The filtering material is considered to be satisfactory.

Results of Tests.—The sample received consisted of about 50 lb. of material most of the pieces being between 1 in. and 2 in. in diameter. No differentiation was made to distinguish between pieces which had remained whole, or those which had resulted from disintegration. In general, the material appeared to be a dense white limestone the fracture of which showed a fine, close crystalline structure.

Determinations of apparent specific gravity, absorption, and porosity, and chemical analyses were made on two samples: (*A*) A composite of chips from a number of large pieces; and (*B*) a composite made by grinding a number of small pieces. This procedure followed the line of reasoning that perhaps the small pieces resulted from disintegration of originally larger pieces, although there was no definite information to show that such was the case. The soundness test and the abrasion test were made on composite samples.

Apparent specific gravity, absorption, and porosity determinations in the sample representing the large pieces gave results as shown in Table 1, Item 21, and the chemical analysis is listed as Item 18 in Table 2. The corresponding results on the composite sample of small pieces are arranged in Table 1, Item 20, and Table 2, Item 19, respectively.

For the soundness test a composite sample of ten pieces weighing 1 335 grammes, were subjected to sodium sulfate treatment for twenty cycles. After the twentieth cycle the pieces were washed, dried, screened through a 1-in. sieve, and again weighed. The material passing through the 1-in. screen amounted to 187 grammes, indicating a loss of 14.0 per cent.

One piece failed at the end of the fourteenth cycle, and after the sixteenth cycle one piece broke into two pieces. All the other pieces showed some signs of breaking down, either by a sloughing off of fines from the surface or by chipping or cracking; yet the pieces cannot be regarded as failures. As the test proceeded, the stratified structure of some of the pieces was brought out, particularly in the piece that failed, as failure occurred by the appearance of many horizontal cracks that continued to spread until the piece itself was entirely disrupted.

The sample for the abrasion test was composed of fifty-four pieces having a total weight of 5 000 grammes. After the required 10 000 revolutions, there were fifty-five pieces more than 1 in. in size having a combined weight of 4 857 grammes. This corresponded to a loss of 443 grammes, or 2.86% wear. A test of the fines showed 70.7% passing a 100-mesh sieve.

Conclusion.—These tests show that the specific gravity of the limestone is fairly uniform, twenty pieces showing extremes of 2.81 and 2.62, and averaging 2.69 (Table 1). The corresponding figures for absorption and porosity varied from 2.66% to 0.59%, and from 6.52% to 1.63%, averaging 1.77% and 4.28% respectively. On the basis of the physical constants and chemical analyses, no significant difference can be noted between the large and small pieces.

The sodium sulfate results indicate that when judged by this test alone the material might be considered questionable. A closer observation of the actual conditions in the filter should prove whether the surface abrasion occurs to an undesirable extent in practice. The abrasion test showed a wear of 2.86%, which is within reasonable limits.

Fitchburg, Mass.

The 2 acres of trickling filters at Fitchburg, Mass., were first placed in operation in October, 1914. The filters are 10 ft. deep and consist of local granite with some trap rock from Quincy, Mass. They were designed originally for a total population of 40 000. At present, the load is estimated at about 43 000 persons, with an average flow of 3 788 000 gal. daily, and experiments have indicated that the filters can well care for a population of at least 66 000.

During the fourteen years of operation they have performed very satisfactorily with a minimum of clogging. All that has occurred has been on the surface, which is due in part to a heavy dark green growth that has been noted to a depth of as much as almost ½ in. at certain times on the stones. However, the pooling has never been serious and can always be eliminated by resting the beds for a few days and stirring the surface with a rake or harrow. The filters have been operated continuously, both winter and summer. Temperatures as low as —11° Fahr. and as high as 97° Fahr. have been needed. There is no indication of surface disintegration. During the first year of operation some fine material washed out into the final settling tanks which are of the plain Dortmund type. The unloading of the filters occurs throughout the spring and summer, with most of it usually taking place in May and June. The suspended matter in the filter effluent is fairly heavy and settles quite well. The filtering material may be regarded as highly satisfactory.

Results of Tests.—A small sample of material taken from the filter surface in 1926 was on hand and this was used for the test because it was known that conditions had not changed since that time. Owing to the small size of the sample, the sodium sulfate test was the only one made. Ten pieces of material weighing 964 grammes, were subjected to this test for twenty cycles. Of this number, three were trap rock and seven were granite.

One piece of trap developed cracks after the sixteenth cycle and broke into three pieces after the seventeenth cycle. One piece of granite, after the twentieth cycle, had a few hair cracks, but was whole. After the completion of twenty cycles there were only 10 grammes not retained on a 1-in. sieve, indicating a loss of 1.04 per cent.

Conclusion.—This material had withstood twelve New England winters when the sample was secured. The fact that one piece registered a splitting under twenty cycles indicates that perhaps for certain types of materials twenty cycles is too severe, particularly where splitting rather than crumbling disintegration occurs.

Gloversville, N. Y.

The 3.06 acres of trickling filters at the sewage plant at Gloversville, were placed in operation in 1912. The filters consist of broken limestone from 1.0 to 2.5 in. in size, in layers 5 ft. deep. The material consists of broken limestone, and was obtained at Mayfield, N. Y., a distance of about 8 miles from the plant.

In 1925 the filters served a population of 24 000, with an average flow of 2 700 000 gal. daily. They have been operated the year round. However, the filters are housed with a removable plank cover during the winter.

An inspection of this bed in 1925 showed that there was some slight disintegration of the stone on the surface, although, in general, the stone appeared to be in good condition. Surface clogging was noted particularly in the fall and spring, but this was due largely to grease and suspended matter carried over from the primary settling tanks.

In this connection it should be stated that Gloversville has a strong domestic sewage containing considerable industrial wastes from the local tanneries. No records are available as to the accomplishment of the filters, but the filtering material as a whole has been considered satisfactory.

Results of Tests.—The sample of limestone from Gloversville was a part of one taken from the surface of the bed in 1925. The material is a dense dark gray limestone. Its fracture reveals a very close and finely crystalline structure.

Determinations of apparent specific gravity, absorption, and porosity were made on individual pieces (Table 1, Item 7), and a chemical analysis was run on a composite sample (Table 2, Item 20). Soundness by sodium sulfate was determined on a composite sample, but because of the small size of the sample no abrasion test was made.

The sodium sulfate test was made on a composite sample of ten pieces weighing 999 grammes. After the twelfth cycle one piece showed a number of fine cracks. This same piece "exploded" after the thirteenth cycle. Another piece showed some corner cracks after the fifteenth cycle and, finally, broke into two pieces after the twentieth cycle. Small cracks were noticeable on one other piece after the nineteenth cycle. The total weight of pieces more than 1 in. in size after twenty cycles was 981 grammes. This represents a loss of 18 grammes, or 1.8 per cent.

Conclusion.—The results show that this material with a specific gravity of 2.79, is denser than the usual run of limestone, and also that the absorption and porosity values averaging 0.33% and 0.89% are low. The soundness test, which developed a failure in thirteen cycles in one piece that apparently had stood up in service for thirteen years, again seems to indicate that any material which will withstand twenty cycles of the sodium sulfate test should be considered satisfactory.

Lincoln, Nebr.

The 2.6 acres of trickling filters in the Lincoln, Nebr., Sewage Treatment Works were placed in operation November 20, 1923. The filtering material consists of 1 ft. of ½-in. to 1-in., and 5 ft. of ¾-in. to 1½-in. quartzite, obtained from Luverne, Minn. The filters were designed for a population of 60 000, and, at present, serve 80 000, with an average flow of 4 385 000 gal. daily. They are operated all year.

There are no indications of any disintegration after five years of service. Some slight pools have been noticed on the surface, but these are attributed to fine compacted material. In this connection it might be noted that the top foot is finer than is usually specified. There are no growths on the surface. The filters unload spasmodically occurring mostly in May and October. There are no final settling tanks. The filtering material is considered highly satisfactory.

Results of Tests.—The sample received consisted of about 100 lb. The pieces seemed to be principally flat and rather smooth. The fracture showed the characteristic finely crystalline waxy luster of quartzite.

Tests for apparent specific gravity, absorption, and porosity were made on individual pieces (Table 1, Item 1). The chemical analyses (Table 2, Item 1), abrasion, and soundness tests, were made on composite samples.

Ten pieces of quartzite weighing 1 493 grammes were subjected to the sodium sulfate test for twenty cycles. There was no sign of disintegration or breaking, nor was there any loss in weight after the twenty cycles.

For the abrasion test seventy-seven pieces weighing 5 010 grammes were used. After the test there were seventy-eight pieces larger than 1 in. The fines amounted to 69 grammes, corresponding to 1.36% of wear.

Conclusion.—The physical constants and the chemical analyses show that the material is a high silica, low absorptive quartzite. Although the plant has been in service only five years, there are indications that the material will stand up indefinitely. Confirmation of this viewpoint is apparent in the result of the sodium sulfate test.

Plainfield, N. J.

The 1.75 acres of trickling filters for the joint Sewage Disposal Plant of Bound Brook, Dunellen, and Plainfield, N. J., were placed in operation in 1916. They have been operated continuously, all the year round. The filters consist of 6-ft. layers of 1-in. to 2½-in. local trap rock. They were designed for a population of 40 000 and, at present, serve approximately 35 000.

No disintegration of the filtering material has been noted and the filters have produced a good effluent. The material is regarded as being highly satisfactory.

Results of Tests.—The sample consisted of about 20 lb. of material taken from the surface of the filter. The stone is a dark gray to brown trap rock with a fracture showing a dense fine structure.

Apparent specific gravity, absorption, and porosity determinations were made on individual pieces (Table 1, Item 10). Chemical analyses (Table 2,

Item 9) and soundness tests were made on composite samples. The abrasion test was not run.

For the soundness test, ten pieces of the material having a combined weight of 1 745 grammes were subjected to sodium sulfate treatment for twenty cycles. One piece which had an earthy appearance developed cracks after the eighth cycle and broke into three pieces after the thirteenth cycle. One other piece broke into two pieces after the fifteenth cycle. Very small pieces split off the corners of two other pieces without further change. Two other pieces showed slight cracks. The weight of material passing a 1-in. mesh after twenty cycles was 19 grammes, indicating a loss of 1.1 per cent. Although this material cannot be regarded as having failed in the test, it is obvious that the sodium sulfate develops a certain weakness which was not apparent formerly.

Conclusion.—The high specific gravity and the low absorption and porosity classify this material as trap rock. The chemical analysis also is fairly typical of basaltic rocks. The sodium sulfate test indicates that materials unaltered by conditions encountered in actual service may be effected and show tendencies to split up, when subjected to twenty cycles.

Cleveland, Ohio

The 6 acres of trickling filters in the Southerly Sewage Treatment Works at Cleveland, Ohio, were placed in operation in 1928. The filtering material consists of 2 to 3-in. limestone placed to a depth of 10 ft. The limestone was obtained from Sandusky, Ohio, and was chosen as being suitable after long exhaustive tests on various materials. The same material was used in the new sewage plant at Akron, Ohio.

Some fine material washed from the filters during the early days of operation, as was to be expected. The plant is of too recent construction to make any observations regarding efficacy of operation or tendency toward disintegration and pooling.

The filters are designed for a population of 240 000 and will be operated on an all-year round basis, with such continuity as the stage of the Cuyahoga River will permit. The final settling tanks are equipped with mechanical devices for the removal of sludge.

Results of Tests.—The sample from Cleveland consisted of about 50 lb. of material taken at random from the surface of the filter. The stone appeared to be quite hard, and its fracture revealed a finely crystalline, light gray structure. Several pieces showed a stratified structure, but the stratification appeared to be close rather than loose.

Determinations of apparent specific gravity, absorption, and porosity were made on several individual pieces (Table 1, Item 12), and the chemical analyses (Table 2, Item 22) and soundness test were made on composite samples.

For the soundness test ten pieces of the stone weighing 2 959 grammes were subjected to sodium sulfate treatment for twenty cycles. Three of the pieces were from those which showed the close stratification. All ten pieces completed the twenty cycles and still remained whole, although the three

pieces that showed close stratification seemed to have a slight tendency to open up along the lamination planes. The total weight of particles more than 1 in. in size after the twenty cycles was 2 956 grammes, showing a loss of fines of 3 grammes, or 0.11 per cent.

Conclusion.—The physical constants and chemical analysis show the material to be a fairly high calcium limestone with low absorption. The sodium sulfate test indicates that for the most part the material will be permanent with a tendency toward weakness in those pieces which show laminated structure. Losses from crumbling at the edges appear to be low.

Rochester, N. Y.

The 1 acre of trickling filter of the Rochester (Brighton), N. Y., Sewage Treatment Works was placed in operation March 1, 1916. The filter consists of broken limestone obtained from Akron, Ohio, and Leroy, N. Y. The material ranges in size from ¾ in. to 1¼ in., and is 6 ft. in depth. The plant was designed to serve 10 000 persons, and at present accommodates 14 700 persons with an average flow of 1 850 000 gal. daily. The filters have been operated all the year with temperatures of the air varying from 10° to 90° Fahr., and of the sewage from 41° to 63° Fahr. Unloading is spasmodic, occurring mostly in May and September. The unloaded material which is fairly heavy, is settled out in an open shallow basin. There are no obvious washings from the filter material in the sludge collected in this basin.

No clogging of any seriousness has occurred. Temporary clogging due to greenish brown leathery growth is relieved by surface raking each year. No appreciable disintegration is noticed on the surface and the material is considered entirely satisfactory.

Results of Tests.—The sample from the Rochester filter weighed about 50 lb. A small carton of unused stone was included. The majority of the pieces appeared to be between 1.0 and 1.5 in. in size. The stone was dense, black, flinty limestone, very hard, and showed a conchoidal fracture.

Apparent specific gravity, absorption, and porosity tests (Table 1, Items 5 and 2, respectively) and chemical analyses (Table 2, Items 10 and 11, respectively), were made on two samples, (*A*) a composite of the used stone, and (*B*), a composite of the unused stone. The sodium sulfate and abrasion tests were run on composite samples representing both used and unused material.

For the soundness test, ten pieces of the filter stone having a total weight of 1 297 grammes were subjected to sodium sulfate treatment for twenty cycles. One flinty piece "exploded" after the tenth cycle. One piece broke into two pieces after the eighth cycle, and four pieces showed fine cracks at the end of the test. These fine cracks were more noticeable in the flint. The total weight of pieces of more than 1 in. in size at the end of twenty cycles was 1 145, a loss of 152 grammes, or 11.7 per cent. This relatively high figure is explained by the fact that the "exploded" piece went almost completely to fines.

Fifty pieces having a total weight of 5 000 grammes constituted the sample for the abrasion test. After the required 10 000 revolutions there were 71 pieces weighing 4 774 grammes. This represents a loss as fines of 231 grammes,

or 4.6% wear. The breaking up is probably accounted for by the brittleness of the flint.

Conclusions.—The physical constants and chemical analyses demonstrate no marked difference between the used and unused stone. The silica indicates roughly the amount of flinty material present. The absorption is very low.

The sodium sulfate test indicates that certain flints which have withstood service for twelve years, can be broken up in less than twenty cycles. Observations on similar material points to the possibility that all flints and cherts do not behave the same, either in actual service or when subjected to test. However, in general, it may be concluded that if the flint will withstand twenty cycles of sodium sulfate, it will also stand up in service.

Columbus, Ohio.

The 10 acres of trickling filters of the Columbus, Ohio, Sewage Treatment Works were placed in operation in November, 1908. The filters consist of 5½ ft. of crushed limestone, the lower 10 in., 2½ to 4 in., and the remainder, ½ in. to 2 in., in size.

The material was obtained from the Columbus and Marion, Ohio, quarries. The filters were designed for a population of 200 000, and, at present, are serving 291 000 persons, with an average flow of 30 000 000 gal. daily. The filters have not been operated in winter. The temperatures of the atmosphere and sewage have been observed to range from —10° to 96° Fahr., and from 44° to 74° Fahr., respectively.

No serious clogging has been experienced although the filters are greatly overloaded in summer. Pools on the surface have been attributed largely to growths. The effluent is settled in shallow plain tanks and the sludge which is rather light and sticky, is said to contain washings from the filtering material.

The intermittent operation of the filter, that is, its shut-down during the winter, obviously re-acts favorably as regards disintegration and clogging difficulties. However, some of the surface material has disintegrated and a portion of this has worked down into the filter. On the whole, the material may be regarded as being only partly satisfactory.

Results of Tests.—The sample from Columbus was made up of material taken from the surface and from the interior of the bed to a depth of 3 ft. There were included, also, wrapped separately, pieces of material which had undergone an obvious breaking up. An inspection of the sample showed that there were two distinct kinds of stone and possibly a third. One kind might be classified as a fine crystalline bluish-gray limestone containing distinguishable fossil remains, while the second kind appeared as a yellowish limestone containing seams of a lustrous colorless mineral, possibly calcite. The third variety was suggested by the fact that there were some pieces which felt decidedly more gritty to the touch, but otherwise resembled the yellowish limestone.

Apparent specific gravity, absorption, and porosity were determined and chemical analyses were run on samples of both sound and disintegrated material. In each case the sample was prepared from chips taken from individual

pieces of sound or broken-down material. The sodium sulfate test was made on a composite sample, one-half of which was of material which had remained whole in service, and one-half of material which had undergone disintegration. The abrasion test was run on a composite sample.

In Table 1, Item 15 is from a sample of sound material; Item 22 is from material partly disintegrated; Item 26, from material considerably disintegrated; Item 25 is a sample of partly disintegrated stone taken from all types of material; and Item 23 is from a composite sample of disintegrated material using samples from Items 22, 25, and 26. In Table 2, Items 2 and 25 are the chemical analyses of the materials described in Table 1, Items 26 and 22, respectively.

For the soundness test, ten pieces of the Columbus material weighing 1 749 grammes, were subjected to sodium sulfate treatment for twenty cycles. Of these ten pieces, five were known to have resulted from disintegration and five had apparently resisted disintegration. Three pieces consisting of cherty or sandy material were removed from the test due to failure, one after the second cycle, one after the tenth cycle, and one after the nineteenth cycle. All these pieces were parts of others that had disintegrated in service. One other of the pieces, resulting from a breaking down on the bed, showed fine cracks at the end of twenty cycles. The remaining six pieces apparently were unaffected after twenty cycles. The weight of pieces more than 1 in. in diameter after twenty cycles was 1 257 grammes, representing a loss of 492 grammes, or 28.2 per cent. This high figure is due to the fact that one piece went almost entirely to fines.

For the abrasion test fifty-one pieces weighing 5 024 grammes were used. After the 10 000 revolutions there were fifty-eight pieces larger than 1 in. in size, and 354 grammes of fines, indicating 7.05% of wear.

Conclusion.—A study of the results of the physical constants shows in general a lower specific gravity and a higher absorption in the pieces resulting from disintegration than in those which had remained sound. The chemical analysis shows the high silica in the cherty material which failed. On the other hand, there appears to be no clue in the chemical analysis of the other sample to suggest failure. The results of the sodium sulfate test, indicates that, with this type of material, twenty cycles should be sufficient on which to base judgment as to the suitability of a material, since 80% of the pieces which had failed in service registered failure under test.

Fort Sheridan, Ill.

The filter bed from which the sample was taken at Fort Sheridan, Ill., was constructed in 1919. Of the original bed, only about 8 640 sq. ft., or 0.2 acre, has been used. The population served is about 2 000.

The filtering material, as originally placed, was a dense type of limestone rock ranging in size from ½ in. to 2½ in., and contained a relatively large quantity of small stone. This material has shown decided evidence of deterioration by breaking down into small pieces. Observation on an unused part of the bed would indicate that breaking down occurs even where no sewage is applied. Apparently the disintegration takes place during the summer as

well as during the winter. Furthermore, large quantities of fine material appear to have been included in the bed originally by improper grading, cleaning, and placing.

At present, the limestone is covered with about 6 in. of granite, where originally there were only about 2 in. The granite has been taken from the unused part and applied to the used section. It is stated that there are signs to indicate that the material in the bed is compacting together or is crumbling down because the surface of the material is constantly lowering. A considerable quantity of sand, including limestone particles, can be found in the outlet conduits.

Results of Tests.—The sample collected was taken to represent as nearly as possible, typical material which had not been affected by exposure. As received, there were about 20 lb. of almost entirely small pieces. The material is a rather white crystalline limestone with a chalky surface. The fracture sometimes showed a bluish mottled appearance, and there seemed to be an appreciable number of large crystals present, judging by the size of the cleavage planes in the fracture. Some pieces also showed the presence of fossil remains.

As it was impossible from the sample to differentiate between stone resulting from disintegration and that resisting disintegration, only one composite sample was examined. Tests for apparent specific gravity, absorption, and porosity were made on individual pieces (Table 1, Item 19), and the chemical analysis (Table 2, Item 24) and sodium sulfate and abrasion tests were made on composites.

For the soundness test, 10 pieces weighing 3 274 grammes were subjected to sodium sulfate treatment for 20 cycles. Two pieces were removed due to failure, one after the tenth cycle and the other after the eighteenth cycle. In addition, one other piece broke into two after the fifteenth cycle. There was evidence of considerable surface abrasion based on the fact that as early as the fourth cycle, the sodium sulfate solution appeared muddy. On examination the pieces had a gritty, sandy "feel". Only two of the pieces completing the twenty cycles appeared to be unaffected. The weight of material larger than 1 in. in diameter remaining after twenty cycles was 2 786 grammes. This represents a loss of 488 grammes, or 14.9 per cent.

The sample for the abrasion test was made up of 157 pieces weighing 5 000 grammes. After the 10 000 revolutions there were still 157 pieces, 1 in., or greater, in size, weighing 4 880 grammes. This corresponds to a loss of 120 grammes, or 2.4% wear. Of the fines, 92.0% passed through the openings of a 100-mesh sieve.

Conclusion.—The physical constants and the chemical analysis show the material to be almost a true dolomite with somewhat low specific gravity and medium absorption. The magnesium appears slightly in excess of that required for a true dolomite, suggesting the presence of some magnesite which may account for the chalky appearance.

There appears to be no relation between the percentage of wear and the soundness. The sodium sulfate test indicates an unsatisfactory stone in twenty cycles. However, it is questionable whether the test indicates the extent of weakness observed in actual service.

Marion, Ohio

The 1.4 acres of trickling filters at the Marion, Ohio, Sewage Treatment Plant were placed in operation in July, 1924. They have been operated both winter and summer as the stage of the Little Scioto River permits. The filters are designed for 40 000 persons and, at present, serve about 23 000, with an average flow of 1 800 000 gal. daily. The filtering material consists of 10 ft. of local broken limestone ranging in size from 1½ to 2¾ in. Pooling on the surface has not occurred to any extent. Until recently, there has been little surface growth.

Some disintegration of the surface material has occurred. In spots, also, some fine material was deposited in the original placing. Compacting occurs in the vicinity of these fines and under heavy loading would probably cause some pooling.

The filters unload seasonally, the bulk occurring in May and June, although some unloading is taking place constantly. The sludge is very light and settles with difficulty in the plain rectangular trough-bottom settling tanks. A continuous light spray on the surface of the tank from revolving sprinkling nozzles greatly assists the settling.

Results of Tests.—About 60 lb. of the sample was received in a box divided into three compartments. One compartment contained stone in good condition from the top of the bed; one contained sound stone from 3 ft. below the surface; and the third contained a number of stones in various stages of disintegration. The stone appeared to be composed of dense bluish limestone with some lighter in color, and having larger crystals. There were specimens also, particularly among the disintegrated pieces, which appeared hard and almost chert-like.

Three samples were prepared for physical and chemical analyses: Chips from sound surface stone; chips from sound stone 3 ft. below the surface; and chips from disintegrated pieces. The soundness and abrasion tests were determined on composite samples, including all three types mentioned.

The physical tests for these three samples are arranged in Table 1 under Items 17, 8, and 14, respectively. The corresponding chemical tests are given in Table 2, Items 21, 23, and 26, respectively.

The soundness test by sodium sulfate was made on ten pieces for twenty cycles. The composite was made up of four pieces resulting from disintegration, and six sound pieces, three from the surface of the bed and three from 3 ft. below the surface. The total weight of the sample was 3 805 grammes. No pieces were removed because of failure. All the sound rock completed twenty cycles apparently unaffected. However, all four of the pieces resulting from disintegration showed slight cracks, although none of them had enough to cause failure. The total weight of material larger than 1 in. in diameter at the end of twenty cycles was 3 778 grammes. This corresponds to a loss in weight of 27 grammes, or 0.71 per cent.

For the abrasion test 47 pieces weighing 5 054 grammes were used. After the required tumbling there were 54 pieces weighing 4 929 grammes, indicating a loss of 125 grammes, or 2.47% of wear.

Conclusion.—The analyses show considerable variation in the composition of the material. Furthermore, there is no apparent relationship between disintegration and absorption, since the average absorption in the pieces resulting from disintegration is less than that in the sound pieces on the surface, the values being 1.16% and 1.39% respectively. It is noted also that the insoluble matter in the sound material is greater than in the disintegrated material. This same relation holds true in that part of the insoluble matter that is nonvolatile with hydrofluoric acid (see Table 2).

In the sodium sulfate test, twenty cycles did not produce a distinct and positive breaking down in those pieces that had resulted from the disintegration of a larger piece in actual service. This indicates that, with certain types of stone, twenty cycles may not be sufficient to foretell its behavior in actual service.

Mason City, Iowa

The trickling filter of the sewage treatment plant of Mason City, Iowa, was placed in operation in 1918. The filter is 1 acre in area and is composed of 7 ft. of 1-in. to 2½-in. crushed local limestone. The filters were designed for a population of approximately 20 000. The sewage is very dilute due to excessive infiltration and, in consequence, the filter at times cannot take the entire flow. It is operated all the year round but is protected from the weather by a covering.

The unloading of the filter occurs continuously and the sludge which is fairly light is settled in an Imhoff type of final tank. Some stone washings from the filter are observed in the final tank sludge. No serious clogging of the filter is reported. There is little growth on the surface.

No disintegration of any consequence has occurred except where the windows in the housing had been left open and freezing took place. For this reason, it is believed by the officials that were it not for the covering the material would break down rapidly.

Results of Tests.—The sample received weighed about 40 lb. The material was separated to differentiate between: (*a*) Material in good condition from beneath the surface; (*b*) material from the surface showing incipient disintegration; and (*c*) small chips of material which had resulted from a breaking down of large pieces. In general, the rock may be characterized as having a light yellow color streaked in many cases with a darker shade; very fine crystals often disseminated throughout larger elongated ones; a conchoidal fracture; and a rather waxy luster. In many cases the rock appeared to be shot through with seams of sandstone, sometimes of sufficient breadth to permit easy identification.

A sample, (*A*), composed of apparently sound rock from beneath the surface was tested for apparent specific gravity, absorption, and porosity (Table 1, Item 13), and chemical constituents (Table 2, Item 27). Another sample, (*B*), of partly disintegrated material of nitrogeneous composition was examined for physical constants only (Table 1, Item 16). Sample (*C*) was part of a piece which crumbled in handling and was analyzed for chemical constituents only (Table 2, Item 6).

The soundness and abrasion tests were made on composite samples. For the soundness test ten pieces of stone weighing 1 723 grammes were subjected to the sodium sulfate test for twenty cycles. Of these, five were pieces of sound stone and five were pieces that had resulted from disintegration. Three pieces of the disintegrated material were removed, due to failure, two after the eighth cycle and one after the fourteenth cycle. A fourth piece of the disintegrated stone which contained a broad sandstone seam appeared to have undergone considerable surface abrasion, although it was not cracked after the twentieth cycle. The fifth piece apparently withstood the twenty cycles without effect. Of the five sound pieces, one broke into two after the twentieth cycle, but the other four apparently were unaffected. All three of the pieces that failed contained sandstone seams, and two of these pieces crumbled almost entirely to fines. The total weight of material more than 1 in. in diameter after twenty cycles was 1 372 grammes, indicating a loss of 351 grammes, or 20.4 per cent.

For the abrasion test forty-eight pieces weighing 5 028 grammes were used. After the required tumbling there were thirty-five pieces more than 1 in. in size, weighing 4 800 grammes. The fines amounted to 228 grammes, indicating 4.54% of wear.

Conclusion.—This material has a relatively low absorption and is of a rather uniform density. Its composition varies from practically a pure limestone to some pieces containing for the most part earthy silicates and silica. The chemical analysis of a piece which crumbled showed a high content of earthy material. Where this value is very high the material often is obviously unsound. Furthermore, there seem to be indications in other investigations that there is a rather definite, or better, indicative relationship between the quantity of earthy material as shown by analysis and the liability to failure in the soundness test. Furthermore, this value often is relatively small. This particular investigation has shown little conclusive evidence in this direction.

In this sample the results of the test apparently support the assumption that twenty cycles of the sodium sulfate test are sufficient to indicate the suitability of a material to withstand disintegration in service.

Pontiac, Mich.

The eight circular trickling filters totaling 1.75 acres of the Pontiac, Mich., Sewage Treatment Works were placed in operation in 1921. A special feature of the filters is the dosing by means of rotating distributors. The filters were designed for 50 000 persons and are now serving approximately 60 000, with an average flow of 6 500 000 gal. daily. The filters are composed of 5.5 ft. of local gravel ranging in size from 1 in. to 4 in. All kinds of material are noted, from soft shales and sandstones to flint and granite. The filters are operated all the year round with extremes of temperatures from —10° Fahr. to 90° Fahr.

During the summer there is little pooling on the surface, but in the spring this is more noticeable, probably due to stored solids. The beds unload regularly, almost always in June. The effluent from the filters is settled in plain tanks of the Dortmund type. Both light and heavy sludge are found, indicating some sand washings from the filter.

Some disintegration noted is confined chiefly to the shales and sandstones. The shales appear to be disintegrated in the body of the bed as well as on the surface. The sandstone disintegration appeared to be restricted chiefly to the surface, those pieces found in the body of the bed appearing to be unaffected. On the whole, this material may be regarded as satisfactory, the disintegration being confined to a relatively small part of the material, and that mostly on the surface.

Results of Tests.—The sample from Pontiac consisted of about 60 lb. obtained by taking small portions from a cross-section about 3 ft. deep. There was a variety of material, including flint, granite, silica, sandstone, feldspar, and shale. A number of disintegrated pieces found on the surface were included separately. These for the most part were sandstones and shales.

Owing to the variety of the material no composite tests were made except for soundness. Two samples; (*A*) of red sandstone, and (*B*) of gray sandstone, both resulting from disintegration, were examined for apparent specific gravity, absorption, porosity, and chemical constituents. A sample, (*C*), of shale was examined for physical constants.

The physical tests for Samples (*A*), (*B*), and (*C*) are listed in Table 1 as Items 28, 24, and 31, respectively; and the chemical tests for Samples (*A*) and (*B*) are listed in Table 2 as Items 3 and 4, respectively.

For the soundness test ten pieces, totaling 2 049 grammes, were subjected to sodium sulfate treatment for twenty cycles. Three pieces were parts of stone which had undergone disintegration in service and seven were pieces which had apparently remained sound. Three pieces were removed from the test due to failure, at the end of the seventh, tenth, and fifteenth cycles. Of these, two were pieces resulting from disintegration on the bed, and the third was an apparently sound sandstone pebble from the interior of the bed. The third of the disintegrated pieces broke in two at the end of the thirteenth cycle. The remaining six pieces passed through the twenty cycles apparently unaffected. None of these was sandstones, whereas all that failed or showed signs of failure were sandstones. The method of failure in all cases was gradual, that is, the stone seemed to swell and gradually become softer and softer until it broke or crumbled entirely. The total weight of material more than 1 in. in size after twenty cycles was 1 545 grammes, representing a loss of 504 grammes, or 24.6 per cent. This high figure is due to the failure of the sandstones, and obviously as in the other tests, does not represent the quantity of failing material which is present in the filter beds.

Conclusion.—The results of the observations and tests of this particular material point to the possibility that shale will disintegrate both on the surface and in the body of the bed. Furthermore, some sandstones may go to pieces on the surface and remain intact beneath.

Again, it is noted that these sandstones have low specific gravities and high iron content. The oxidation of the iron may have an important part in the disruption, although if this were the case, one would expect the breaking down throughout the bed. The soundness test indicates that twenty cycles of sodium sulfate will reveal probabilities of unsoundness in sandstone material.

Reading, Pa.

At the Reading, Pa., Sewage Treatment Works, there are four trickling filter units, each 1 acre in area. The first of these units (No. 4) containing 5 ft. of broken slag, was put in service in January, 1908. Another unit (No. 2) containing 6 ft. of broken feldspar was put in service late in 1909. The third unit (No. 1) was completed in the autumn of 1911 and closely resembles the first unit (No. 4). A fourth unit (No. 3), also consisting of 5 ft. of broken slag, was constructed in 1915. These filters have been operated continuously since they were installed.

An examination of the filters in the early spring of 1928 showed that in Nos. 4 and 1, the material appeared quite well reduced to small sizes. In this connection it should be stated that the slag in these filters when originally placed, was very poorly sized (1½ to 6 in.), and contained numerous pockets of fine material. Bed. No. 3, on the other hand, was made up of well graded slag from 1½ to 4 in. in size.

All the beds showed that some disintegration had taken place on the edges not wetted by sewage, due to natural weathering. This was noticeable but not pronounced. Local officials are of the opinion that the breaking down throughout the beds has been progressive through the years. Beds Nos. 4 and 1 indicate that the fine material is working down toward the bottom. Comparatively little fine material was found in Filter No. 3, the most recent bed. Filter No. 2 composed of feldspar appears rather well broken down.

Filter No. 4, the oldest bed, appeared to have the most fine material and was quite dirty all the way down. None of the beds seemed to be waterlogged. Many worms were found in all the filters. Relatively little pooling was seen and surface growths were sparse. Each bed had its own final settling tank. These tanks are cleaned monthly during the summer. The most sludge is removed in May and June, about two to two and one-half times as much as during the other months. The sludge is said to contain some washings from the filter material. The final effluent was of fair appearance.

Results of Tests.—Samples of about 25 lb. each were taken from Filters Nos. 4, 1, and 3, by digging about half way down into the bed. These were boxed separately. The sample from Bed No. 1 as received, seemed to contain the most fine material, and two samples of this were examined: (*A*) a composite of small pieces; and (*B*) a composite of larger pieces. The results are shown in Table 1, Items 30 and 18, and in Table 2, Items 12 and 14

For the soundness test ten pieces from Bed No. 1, weighing 1 055 grammes were subjected to sodium sulfate treatment for twenty cycles. These were made up of four small and six large pieces. One of the large pieces, dense slag, was removed after the fifteenth cycle, having broken into three pieces. One of the small pieces showed excessive surface abrasion. Two other pieces broke in two after the fifteenth cycle. The other six pieces completed the twenty cycles, apparently in good condition, although surface abrasion in all cases was distinct. The total weight of material of more than 1 in. after the twenty cycles was 939 grammes. This corresponds to a loss of 116 grammes, or 11.0 per cent.

For the abrasion test on material in Bed No. 7, 110 pieces more than 1 in. in size and weighing 4 600 grammes were used. After the required tumbling there were found to be only 109 pieces of more than 1 in. The fines, that is, the material passing the ½-in. mesh, weighed 676 grammes, showing 14.7% of wear. There was also much small material between ½ in. and 1 in.

Composites from Beds No. 4 and 3 were also examined. In each sample apparent specific gravity, absorption, and porosity were determined and a chemical analysis was made. Tests for abrasion and soundness were made on a single composite from each bed. The tabulated results are arranged in Table 1, Items 27 and 29, and in Table 2, Items 13 and 15, respectively.

For the soundness test on material from Bed No. 4, ten pieces weighing 902 grammes were subjected to sodium sulfate treatment for twenty cycles. Two pieces were removed for failure, one after the tenth cycle and the other after the thirteenth cycle. The first was a dense piece, the second of light porous structure. Two other pieces broke in two, one a small piece after the fifteenth cycle, and the other an exceedingly porous piece, after the nineteenth cycle. Another piece of rather porous material showed considerable evidence of surface abrasion. The total weight of material larger than 1 in. after twenty cycles was 795 grammes. This represents a loss of 107 grammes, or 11.9 per cent.

For the abrasion test on material from Bed No. 4, 111 pieces larger than 1 in. in size and weighing 4 748 grammes were used. After the required tumbling there were only 103 pieces larger than 1 in. There was a great deal of small material between ½ in. and 1 in. The fines passing the ½-in. mesh equalled 711 grammes, indicating 15% of wear.

For the soundness test on material from Bed No. 3, ten pieces were subjected to sodium sulfate treatment for twenty cycles. The total weight of the sample was 1 373 grammes. All ten pieces completed the test apparently unaffected. The total weight of the material at the end of the twenty cycles was 1 346 grammes, corresponding to a loss in weight of 27 grammes, or 1.95 per cent.

An abrasion test was made on a sample of fifty pieces from Bed No. 3 weighing 5 000 grammes. After the required 10 000 revolutions, there were fifty-four pieces more than 1 in. in size weighing 4 122 grammes. This equals a loss in weight of 878 grammes, or 17.55% wear. The material which broke off during the test was very fine, as a mechanical analysis showed that 96% of it passed the 100-mesh screen.

Conclusion.—The results of the examination of the slag in the Reading filters show the wide variation in the physical constants of this material. The specific gravity, absorption, and porosity values were found to vary between 2.83 and 1.53; 13.35% and 0.31%; and 18.65% and 0.79%, respectively. On the other hand, there were no wide differences in the chemical analyses of the four samples, except that the iron content in the small pieces from Filter No. 1 was greater than in the others. It is worthy of note also that the absorption figure in the small disintegrated fragments of Filter No. 1 was greater than in the other samples, except in the good material of Filter (B), where the values were 6.37% and 6.35%, respectively.

The results of the soundness test suggest that the sodium sulfate treatment for twenty cycles will indicate a material liable to disintegration in service, as shown by the results on Filters Nos. 1 and 4. Furthermore, the test on Filter No. 3, where the material has apparently stood up well for thirteen years, shows that the percentage of wear as found in the abrasion test must be interpreted with care.

Schenectady, N. Y.

The 3 acres of trickling filters of the Sewage Treatment Works of Schenectady, N. Y., were placed in operation in January, 1915. They have been run continuously all the year round with variations in the temperature of the atmosphere between —10° Fahr. and 97° Fahr., and of the sewage between 42° Fahr., and 68° Fahr. At present, a population of approximately 65 000 is served with an average flow of 8 500 000 gal. daily. The filtering material consists of 5 ft. of 1½ to 2½-in. broken limestone obtained from Cobleskill, N. Y.

There has been some surface clogging in the past, due to surface growths and matter carried over from the tanks, but this has been eliminated in recent years. Some disintegration is noted on the surface, and some small material is found in the body of the bed. There are no final settling tanks. Above certain stages of the Mohawk River the final effluent must be pumped. The records show that the filters produce a good effluent.

Results of Tests.—The sample received from Schenectady was divided into three parts: Sound stone from beneath the surface; sound stone from the surface; and disintegrated stone from the surface. The material for the most part is a dark gray hard limestone more or less stratified in structure. The seams between the strata are darker than the stone itself, and appear to be filled with carbonaceous matter.

Two samples were examined for physical constants and chemical constituents: (*A*) A composite of the sound stone, and (*B*), a composite of the disintegrated material. (See Table 1, Items 6 and 11, and Table 2, Items 17 and 8, respectively.) The soundness and abrasion tests were made on composites.

For the soundness test ten pieces of stone having a total weight of 2 359 grammes were subjected to the sodium sulfate test for twenty cycles. Of the ten pieces two were taken from the disintegrated surface stone; two from sound surface material; and six from the sound pieces obtained beneath the surface. Six pieces of the good stone completed the twenty cycles apparently unaffected. Two pieces of the sub-surface material showed surface abrasion, noticeable particularly in one piece after the eleventh cycle and in the other after the fourteenth cycle. The pieces resulting from disintegration in service completed the twenty cycles without failure, but showed signs of chipping, one after the tenth and one after the fourteenth cycle. After the twentieth cycle both showed fine cracks along lamination planes. The weight of material larger than 1-in. after the twenty cycles was 2 347 grammes, a loss of 12 grammes, or 0.5 per cent.

For the abrasion test fifty pieces weighing 5 000 grammes were used. After the required 10 000 revolutions there were fifty-three pieces having a

diameter of 1 in. or more, the total weight of which was 4 880 grammes. This represents a loss due to abrasion of 120 grammes, or 2.4% wear. A mechanical analysis showed that 61.1% of the fines passed a 100-mesh sieve.

Conclusion.—The tests show the limestone to be particularly dense and non-absorptive. Furthermore, they seem to indicate again that low absorption (0.86%) is no guaranty against disintegration, although the absorption figure in the sound material was even less (0.32 per cent). The chemical analysis indicates considerable earthy matter in the pieces which have disintegrated, while the abrasion test, from the fact that only 61% of the fines pass a 100 mesh, point to the brittleness of the material.

The sodium sulfate test is not so conclusive as to the information revealed in twenty cycles, although the tendency is to show that twenty cycles are sufficient.

Springfield, Mo.

The Doling Park Sewage Treatment Plant, at Springfield, Mo., was placed in operation in 1912. The trickling filter consists of 0.16 acre of broken local limestone. The plant was designed for 3 500 persons and at present (1928) serves 4 000, with an estimated dry-weather flow of 360 000 gal. daily, which reaches a maximum rate of approximately 900 000 gal. daily during rains.

There are no pools and very little surface growth. The final effluent is settled in rectangular earth tanks and the sludge is said to be light, with no indication of washing from the filter material.

The filters are operated continuously. Only a slight weathering of the stone has been observed on the surface, the disintegration occurring where the rocks are laminated. There are no indications that appreciable quantities of fine material have worked down into the filter. The stone has been considered satisfactory for filtering material.

Results of Tests.—In the sample, which consisted of about 70 lb., no attempt was made to differentiate between sound and disintegrated pieces. The sizing was fairly uniform. The fractures showed a white limestone containing rather large crystals. Fossil remains were found in some pieces. Chert also seemed to be of fairly common occurrence.

Tests for apparent specific gravity, absorption, and porosity, were made on several pieces of the material (Table 1, Item 9). The chemical analyses (Table 2, Item 16), soundness, and abrasion tests were made on composites.

For the soundness test, eleven pieces aggregating 2 305 grammes were subjected to the sodium sulfate test for twenty cycles. A piece of chert was removed due to failure after the seventh cycle. This piece developed a number of fine radial cracks before failure. One other piece broke into two after the tenth cycle. In the case of four other pieces, some of the larger crystals showed a slight spalling. In two other pieces cracks appeared in one case after the tenth cycle and in the other after the fourteenth, but both pieces completed the test without parting. Three pieces went through the twenty cycles without apparent effect. The total weight of material more than 1 in. in size, after the twenty cycles, was 2 271 grammes. This represents a loss of 34 grammes, or 1.47 per cent.

For the abrasion test fifty pieces weighing 5 034 grammes were used. After the required tumbling there were fifty-seven pieces more than 1 in. in size. There was considerable small material between 1 in. and ½ in. The fines amounted to 423 grammes indicating 3.42% of wear.

Conclusion.—The results of the examination of the material show it to be a practically pure limestone containing considerable silica, but little earthy matter. The absorption is very low, and the soundness satisfactory. The sodium sulfate test in this instance seems to be rather too severe than not severe enough. However, if the record of service were not available to certify the satisfactory quality of the material, it could be passed on the basis of the results obtained in the tests.

Interpretation of Results

Apparent Specific Gravity, Absorption, and Porosity.—The average values for apparent specific gravity (48-hour), absorption, and porosity, are shown in Table 1. The porosity figure is a function of both the specific gravity and absorption.

A re-arrangement of Table 1 in a descending order of specific gravity, readily classifies trap rock, granite, dense limestone, quartzite, sandstone, and slags. There seems to be a slight relationship between the specific gravity of limestones and their disintegrating qualities. It will be noted that all stone with an average specific gravity of more than 2.67 was found to be sound, yet there were sound limestones with specific gravity as low as 2.60. On the other hand, the average specific gravity of the disintegrated limestone was generally less than 2.60. It may be stated, therefore, that while the tendency seems to prevail that a dense limestone with specific gravity of more than 2.67 will be sound, this cannot be counted upon as a certainty.

On the other hand, limestones with low specific gravities, say less than 2.50, must be regarded with suspicion, since when this occurs there is likely to be an undesirable quantity of earthy material or sandstone. An exception to this seems to prevail with the slags, although, in general, the slags as well as the low specific gravity limestones show a decided tendency toward surface abrasion.

TABLE 3.

Place.	Absorption (Percentage by Weight).	
	Sound stone.	Disintegrated stone.
Columbus, Ohio	1.17	2.55
Marion, Ohio	0.97	1.16
Mason City, Iowa	0.99	1.34
Schenectady, N. Y.	0.32	0.86

A re-arrangement of Table 1 in an ascending order of average porosity would result in only very slight changes since, in general, a low absorption means low porosity. A study of Table 1 shows that, in general, dense limestones, trap rocks, granite, and quartzite have absorption figures of less

than 1.25 per cent. Some material which disintegrates shows low absorption, particularly certain of the cherty materials. Where sound and disintegrated stone came from the same bed, the relationship shown in Table 3 may be observed as regards the average absorption.

It might be concluded, therefore, that a dense limestone with absorption less than 0.75 may be regarded as being satisfactory. While this may be true, particularly if the highly silicious limestones are excluded, it must be borne in mind that material with higher absorption figures apparently remain sound. This is true of certain limestones as well as the slags, because an examination of Table 1 shows that apparently good material with the following absorption values were found.

Place.	Absorption, (in percentage by weight).
Marion, Ohio	1.39
Fort Sheridan, Ill.	1.63
Decatur, Ill.	1.82
Reading, Pa., Bed No. 3	6.35

In general, therefore, it may be stated that finely crystalline, uniform, low-silica limestones, having a specific gravity of 2.67 or more, and an absorption of less than 1.10%, may be expected to resist disintegration. On the other hand, if the specific gravity is less than 2.67, or the absorption is greater than 1.10, there is no certainty that the material will break down. This appears applicable, particularly to slags.

TABLE 4.—ABRASION OF TRICKLING FILTER MATERIAL.

Item No.	Place.	Material.	Wear (percentage by weight).	Remarks.
1	Lincoln, Nebr.	Quartzite	1.36	Composite sample
2	Baltimore, Md.	Trap and granite	2.34	" "
3	Schenectady, N. Y.	Limestone	2.40	" "
3	Fort Sheridan, Ill.	Limestone	2.40	" "
4	Marion, Ohio	Limestone	2.47	" "
5	Decatur, Ill.	Limestone	2.86	" "
6	Mason City, Iowa	Limestone	4.54	" "
7	Rochester, N. Y.	Limestone	4.60	" "
8	Columbus, Ohio	Limestone	7.05	" "
9	Springfield, Mo.	Limestone	8.42	" "
10	Reading, Pa.	Slag	14.70	Bed No. 1, 1911
11	Reading, Pa.	Slag	15.00	Bed No. 4, 1908
12	Reading, Pa.	Slag	17.55	Bed No. 3, 1915

A study of Table 4 shows that the dense limestones, traps, granite, and the hard quartzite show a relatively low abrasion loss. The Ohio State Highway Department specifies that a Grade A stone shall not have more than 6% of wear. It would appear that some material with a relatively low abrasion loss would prove only partly satisfactory, particularly from a disintegration standpoint, whereas others might be regarded as satisfactory where the abrasion is high. This seems to be especially true of the slags.

CHEMICAL ANALYSES

A study of Table 2 reveals no outstanding characteristic of the material with particular reference to any one constituent. As is to be expected, the

igneous materials, such as granite, trap rock, and quartzite, are high in silica, and, in general, also these materials have low absorption and high specific gravity. Sandstones which have a high silica content, also, appear subject to abrasion and disintegration. However, they usually have a relatively high absorption and a low specific gravity. On the other hand, certain flints and cherts with high silica, low absorption, and normal specific gravity are likely to "blow up" or "shatter" under test, or in service.

In certain types of limestone, which in reality may be calcareous shales with the earthy matter disseminated evenly throughout the entire mass, a study of the insoluble matter, not silica, is of importance. The present investigation throws little light on this phase of chemical analysis, but experience in other investigations seems to point to a relationship between earthy material and likelihood of disintegration, especially disruption by hydrolysis. This requires further study before definite conclusions can be drawn.

In slags, it would seem that the amount of sulfur and of iron oxide would be significant. Sulfur in slag due to the highly reducing conditions in the blast furnace, is present mostly as sulfide combined with one of the basic elements, most probably as calcium or magnesium sulfide. Both these compounds re-act slowly with water to form hydrates and gaseous hydrogen sulfide. This might be considered in much the same light as slaking. The greater the amount of sulfur, the greater is the possibility for disruption. It will be noted that the sulfur content in Beds Nos. 1 and 4 at Reading, Pa. (Table 2, Items 12, 13, and 14) is considerably greater than in Bed No. 3 (Item 15) in which disintegration was noted only to a minor extent, whereas in the other two beds, breaking down of the material was more pronounced.

Observation shows that where iron "shot" is present in a piece of slag, oxidation and hydration will soon effect a swelling and utter disruption of the fragment. In the Reading results it will be noted that the small pieces in Bed No. 1 (Table 2, Item 12), assumed to have occurred from a breaking down of the larger pieces, showed a considerably higher iron content than the material from the other beds. It might be stated, therefore, that the iron oxide content of a slag should preferably be less than 1 per cent.

Therefore, with limestones and dolomites the chemical analysis reveals little of positive value as regards disintegration, although there apparently is some relationship between the quantity of earthy matter, especially when evenly disseminated, and the possibility of disruption. With slags the amount of sulfur and iron oxide appears to have some significance as regards the permanence. Limiting values of 1.00% for iron oxide and of 2% for sulfur as calcium sulfide in slag would seem to be reasonable. Past experience would indicate also that a slag will be more likely to resist disintegration if the calcium oxide is less than 42% and the magnesium oxide more than 5 per cent.

Soundness by Sodium Sulfate

Filtering materials when subjected to the sodium sulfate test for soundness will either remain unaffected, or will suffer a breaking down in varying modes and degrees similar to those occurring in actual service. In the present investigation an attempt was made to evaluate the sodium sulfate test in

respect to the number of cycles necessary to determine the suitability or non-suitability of a given material.

In all about 171 pieces of filtering material were subjected to the sodium sulfate test. Of these, 17 pieces were known to be parts of material which had broken down in service, whereas the remaining 154 pieces had apparently resisted disintegration in service.

TABLE 5.—EFFECT OF SODIUM SULFATE ON DIFFERENT TRICKLING FILTER MATERIAL.

Kind of material.	Number of pieces.	Condition at End of Cycle. 5 Failures.	5 Slight cracks, surface abrasion, spalls, etc.	5 O. K.	10 Failures.	10 Slight cracks, surface abrasion, spalls, etc.	10 O. K.	15 Failures.	15 Slight cracks, surface abrasion, spalls, etc.	15 O. K.	20 Failures.	20 Slight cracks, surface abrasion, spalls, etc.	20 O. K.
Sound Material at Start of Test.													
Granite	10	0	0	10	0	0	10	0	0	10	0	0	10
Gravel	7	0	1	6	0	1	6	1	0	6	1	0	6
Limestone	77	0	3	74	1	11	65	5	28	44	6	32	39
Quartzite	10	0	0	10	0	0	10	0	0	10	0	0	10
Slag	30	0	1	29	1	8	21	3	6	21	3	6	21
Trap	20	0	0	20	0	4	16	1	4	15	2	6	12
Total	154	0	5	149	2	24	128	10	38	106	12	44	98
Pieces of Material Which Disintegrated in Service.													
Limestone	14	1	7	6	3	6	5	4	7	3	6	6	2
Sandstone	3	0	2	1	1	2	0	2	1	0	2	1	0
Total	17	1	9	7	4	8	5	6	8	3	8	7	2
Grand total	171	1	14	156	6	32	133	16	46	109	20	51	100

The results of the tests are summarized briefly in Table 5, according to the kind and state of the material tested. Five rather significant conclusions are indicated, as follows:

(1) Both slag and stone will break down when subjected to the sodium sulfate test for soundness.

(2) Some material which has apparently resisted disintegration in ten years or more of service will register failure in twenty cycles, and sometimes less.

(3) Material which has undergone disintegration in service is more susceptible to the sodium sulfate test than material which has stood up in service.

(4) The destructive effect of the test is not always wholly apparent from observation, particularly in respect to the crumbling and abrasion of fines from the surface and edges.
(5) The type of failure varies greatly from a simple splitting into two pieces to complete destruction by shattering or crumbling into fines.

Two of the essentials to be considered in establishing a soundness test procedure by the sodium sulfate test are: First, how many cycles shall be required; and, second, what constitutes failure.

This investigation shows rather conclusively that five cycles are not sufficient to indicate the permanence of a substance to be used for filtering material. In certain cases, where material has resisted disintegration, twenty cycles appear to be too severe in that these same pieces broke down in the test. However, when the "break-down" occurs as a splitting into three or more distinct pieces, the explanation may be found in the presence of very fine seams. When the disruption is complete, the cause may be either the severity of the test, or perhaps that particular fragment may have come from beneath the surface, where a certain amount of protection is afforded.

A study of Table 5 shows that of the sound pieces only 8% failed in twenty cycles, whereas 47% of those which broke down in service failed in test in twenty cycles or less. Furthermore, there was more spalling, chipping, abrasion, and other miscellaneous minor effects in the disintegrated pieces than in the sound pieces. This is shown by the fact that, of the sound pieces, after twenty cycles 57% remained unaffected, whereas in the disintegrated pieces only 12% were unaffected.

In view of all the circumstances, it appears reasonable to conclude that a satisfactory material should withstand twenty cycles of sodium sulfate treatment without appreciable destructive effect.

As stated previously in this Appendix, breaking down of a filtering material occurs in various ways. In the 1927 progress report it is stated* that:

"A piece of material which exhibits disintegration, marked checking, or cracking, or is split into three or more separate pieces, or is so cracked that it is evident that it will do so, shall be considered to have failed in this test."

It is apparent that all filtering materials are subject to some breaking down. With some, this is insignificant and hardly noticeable, whereas with others it is appreciable and clearly distinct. Furthermore, all the material from some sources is not uniform, some of it being satisfactory and some admittedly unsatisfactory. From the standpoint of economy as well as of operation, it is essential that tolerances, or the amount of relaxation permitted in the use of inferior material, be established. This phase of the matter is now being studied.

In making the sodium sulfate test for soundness, not only should there be careful observations as to the form and extent of any breaking down, but also there should be a record of the loss in weight of the various pieces. As a tentative procedure it is suggested that the weight of all material more than 1 in. in size be recorded, and also the weight of fines less than ¼ in.

* *Proceedings*, Am. Soc. C. E., April, 1928, Society Affairs, p. 270.

From the evidence gathered in this investigation as well as from experience gained in other testing work, it would seem that, in general, twenty cycles of the sodium sulfate treatment are sufficient to establish whether or not a filtering material will break down in actual service. Furthermore, in view of the variations which are likely to occur it is suggested that, unless failure occurs before that time, any test of material be continued for twenty cycles.

As a check on the usableness and practicability of the sodium sulfate test, samples of slag and limestones, identical as far as possible, have been sent to about thirteen testing laboratories where the same procedure will be followed for twenty cycles. It is expected that the results obtained will go far toward establishing the value of the test, and setting up a standardized procedure for general use.

Tentative Conclusions Regarding Testing Filter Materials

The investigation work of 1928 on trickling filter material which has been in service tends toward the conclusions that:

(1) Specific gravity determinations are of value principally in classifying the material as to its origin.

(2) Limestones with specific gravity of less than 2.60 must be viewed with suspicion.

(3) While no hard and fast rule is applicable as regards the relationship between disintegration and absorption and porosity, it may be stated that, in general, a sound material will have a low absorption figure. On the other hand, certain low absorptive materials, such as some cherts, disintegrate badly, while other materials with relatively high absorption, as some slags, resist disruption.

(4) The abrasion test apparently gives no information relative to the probabilities of disintegration.

(5) The chemical analyses are important principally in evaluating the effect of earthy material, iron, and sulfur, and, in some cases, lime and magnesium. Taken alone, without observation, or other test, the chemical analyses appear to be of little value in predicting disintegration.

(6) The sodium sulfate test appears to be a practical means for determining the suitability of a filtering material, particularly as regards its ability to resist disintegration. More definite specifications as to what constitutes failure in test appear desirable, and this, together with allowable tolerance or relaxation in respect to inferior material, is now under further investigation.

(7) The information gathered from this and other similar investigations indicates that with few exceptions, twenty cycles of the sodium sulfate test are sufficient and necessary to establish the suitability of a filtering material, based on its ability to resist disintegration.

It is appreciated that this investigation has not covered methods of determining whether or not materials will cement and consolidate. Furthermore, no means for ascertaining the ability of a material to slough off the stored solids has been established.

Viewed from the standpoint of disintegration, it may be concluded, therefore, that while all tests give a certain amount of valuable information, the sodium sulfate test apparently offers the best means of determining, with any degree of assurance, the suitability of any given material for use as a filtering medium in sewage trickling filters.

APPENDIX II

Co-Operative Study of the Sodium Sulfate Soundness Test for Sewage Filter Materials by Thirteen Materials Testing Laboratories

The experience of the Committee, confirmed by a number of testing engineers working independently, has led to the conclusion that the sodium sulfate soundness test is the most indicative for predetermining comparative merits of a number of proposed sources of filter material with reference to probable disintegration when placed under conditions of service in sewage trickling filter installations. The laboratory procedure involves a number of details likely to affect the results of the test. To determine the probable reliability of the test, the Committee considered it advisable to send identical samples of limestone and of slag to a number of selected laboratories properly qualified and equipped and to have the sodium sulfate test applied according to a detailed laboratory procedure as determined by the Committee.

Thirteen different laboratories have offered to co-operate in the program of soundness tests, principally because of their interest in this or similar material analyses. Samples of limestone were obtained, including duplicate pieces of five different qualities of material. These have been sent to the several co-operating laboratories through the courtesy of The France Stone Company, of Toledo, Ohio. Likewise, similar samples of slag have been sent to the several co-operating laboratories through the courtesy of Mr. Fred Hubbard, Consulting Engineer for The Standard Slag Company, of Youngstown, Ohio.

The sets of samples were sent to the several laboratories in duplicate and the selection of the samples was made so that the set sent to each laboratory would be as nearly identical with all other sets as was possible. The pieces of limestone were broken from the same block of stone for each grade of material and the pieces of slag were selected from the same run of material.

Emphasis in this series of tests has been placed upon the details of the laboratory procedure. The tests are in no sense a comparison between the several materials being used. In order that more useful information may be obtained and that the test shall not be prolonged beyond twenty cycles, it was thought best to select both the limestone and slag so that some of the pieces in each sample might fail under the test.

The details of the test procedure have been furnished to each of the co-operating laboratories with the request that the soundness tests be conducted following the detailed instructions. The laboratory procedure was based on considerable experience in the testing of numerous samples of filter material. A copy of the instructions, together with an accompanying letter, is attached hereto.

It is thought that the experience of such a large number of testing laboratories should be a relatively severe test of the practicability of the tentative procedure for making the soundness test as outlined by the Committee. Accordingly, each of the co-operating laboratories has been requested to discuss in detail the procedure outlined. These discussions, with the results

of the tests and a description of the methods used in handling, should enable the Committee to develop a well studied procedure for the sodium sulfate soundness test, as it relates to the testing of filter material.

The several co-operating laboratories may be classified as follows:

Four commercial materials testing laboratories.
Three State road materials testing laboratories.
One testing laboratory of the crushed stone industry.
One testing laboratory of the slag industry.
One testing laboratory of a company supplying both limestone and slag.
One laboratory at the Canton, Ohio, Sewage Treatment Plant.
The Bureau of Standards laboratory.
Mr. Gascoigne's testing laboratory.

For the most part the several co-operating laboratories have been able to get the tests under way in accordance with the procedure prescribed by the Committee. A few questions have come up relating to minor deviations from the procedure to better fit the routine of the particular testing laboratory. The matter of the temperature of the sodium sulfate solution and the temperature at which the test pieces are dried, appears to be of importance. Photographs are being taken of the test samples before and at the end of the test.

Procedure for Determining Soundness of Filtering Material by Sodium Sulfate

Definition.—By soundness of a filtering material is meant its ability to withstand the conditions of service without undergoing disintegration, or splitting up into small pieces.

General Procedure.—Select not less than eight (8) pieces of the material of the same size to be used on the work. Dry the sample at 100 to 105° cent. for 4 hours. Weigh and identify each piece by number. Record the weights of the individual pieces as well as the total.

Immerse the dry weighed sample in a solution of sodium sulfate saturated between 30° cent. (86.0° Fahr.) and 35° cent. (95° Fahr.), and contained in an agate-ware or other suitable container (preferably with cover) of adequate capacity. A 3 or 4-quart agate-ware kettle or dish serves acceptably.

Allow the immersed material to stand at room temperature, or as near 21° cent. (70° Fahr.) as practicable during the entire test. Crystals of the sodium sulfate should be present at all times in the dish to assure a saturated solution.

After the materials have been immersed for twenty (20) hours remove the pieces one by one, examine them for fracture and incipient cracks, and record their condition. Then, place the entire sample contained in a shallow pan or other suitable protecting device in a drying oven for 4 hours maintained at 100° cent. to 105° cent.

At the end of the 4-hour drying period, remove from the oven and immediately re-immerse in the sodium sulfate solution. One immersion, one examination, and one drying constitutes a cycle.

Repeat this treatment for twenty cycles, or until obvious failure, when the identity of the individual piece is lost.

At the end of the twenty cycles, wash the material thoroughly, boiling if necessary to remove all contained sodium sulfate, saving all fines. Dry and weigh each piece as circumstances will permit, recording the weight of all material more than one (1) inch in size, and calculating the loss of material under one-quarter ($\frac{1}{4}$) inch in size.

Failure.—A piece of material which exhibits disintegration, marked checking, or cracking, or is split into three or more separate pieces, or is so cracked that it is evident that it will do so, shall be considered to have failed in the test.

If for any reason the continuity of the testing must be interrupted, as, for example, over week ends, it is suggested that the sample remain in the solution. A suggestion for recording the results is appended hereto.

In this test use all the pieces received. The slag and limestone should be tested in separate containers. All other conditions should be in parallel.

Letter of Instructions Sent to Each Testing Laboratory Co-operating in the Soundness Test

"Gentlemen.—In accordance with arrangements made by the Committee on Filtering Materials of the Sanitary Engineering Division of the American Society of Civil Engineers, we are sending by express, from Toledo, Ohio, a box of limestone containing two samples of ten pieces each, one sample to be subjected to the sodium sulfate test for soundness, and the other to be used as you wish. Also, it will be noted that five types of stone are represented, two pieces of each type, as follows:

Brown banded, marked 1 and 2.
Uniform brown, marked 3 and 4.
Mottled, hard, marked 5 and 6.
Dense, flinty, marked 7 and 8.
Mottled, soft, marked 9 and 10.

The duplicate samples are similarly marked with the subscript (a), as 1_a, 2_a, etc.

"In some cases the marking may be indistinct, but can be determined from the above schedule. Kindly preserve the original numbering all the way through. Each piece should be weighed before testing, as this has not been done.

"A similar sample of slag will be sent by Mr. Fred Hubbard, of Youngstown, Ohio.

"In order that the procedure may be uniform in all cases, kindly adhere to the directions enclosed herewith. However, there will be undoubtedly some points in the procedure and technique upon which you may have suggestions. We shall be pleased to receive these and consequently solicit your comments.

"Before the pieces are subjected to test will you kindly have taken a 5 by 7-in. photograph as near half size (linear measure) as possible. After the twenty cycles are completed it is desired to have another photograph taken by the same camera under same conditions as to size, arrangement of pieces, etc., in order that the effects of the test may be visualized. The slag and limestone samples should be photographed separately and the total cost should be kept under $10.00 if possible.

"After the final photographs are taken, kindly wrap each piece in paper, either as a whole or in pieces, box in the same container as received, and ship to George B. Gascoigne, 1149 Leader Building, Cleveland, Ohio.

"We appreciate greatly your co-operation in making these tests, and will await with much interest a tabulation of the results obtained by the different laboratories.

"Very truly yours,

COMMITTEE ON FILTERING MATERIALS,

By George B. Gascoigne."

SANITARY ENGINEERING DIVISION
AMERICAN SOCIETY OF CIVIL ENGINEERS
COMMITTEE ON FILTER MATERIALS FOR SEWAGE AND WATER WORKS

Suggested Form for Reporting Soundness Test.

Sample of———————

Soundness by Sodium Sulfate

Tested by———————

	Weight, in Grammes: Number and Condition of Pieces.										
Number	1.	2.	3.	4.	5.	6.	7.	8.	9.	10.	Total.
Weight at start											
Cycles											
1											
2											
3											
4											
5											
6											
7											
8											
9											
10											
11											
12											
13											
14											
15											
16											
17											
18											
19											
20											
Weight of pieces of more than 1-in.											
Loss, percentage.											
Weight of pieces of less than ¼-in.											
Loss, percentage.											
Remarks.											

APPENDIX III

Fine-Grained Filtering Material for Sewage Works

Investigation of fine-grained filter materials during 1928 has been somewhat retarded in favor of the comprehensive study of trickling filter materials. Some additional information has been collected by correspondence with State Sanitary Engineers and interested practicing engineers. However, this has not been sufficiently extended to warrant any definite conclusions at the present time.

The information on fine-grained filter material for sewage work indicates that the use of sewage and filters is confined to a few States and that at present very few new installations are being constructed. However, the Committee feels that the use of intermittent sand filters will continue to play an important part in the art of sewage disposal. It is probable that few, if any, large installations will be made, and that the use of sand filters will be confined to small plants; but the fact that they enable a "dressing up" of the effluent which cannot be obtained from other plants, will cause them, it is believed, to be used wherever great refinement in purification is required. It is probable that as demands for more complete purification increase, sand filters may become even more important than they have been in the past. There are a number of plants at present in which sand filters are used in addition to other secondary purification works, such as activated sludge, etc.

Considerable investigational work has already been done on fine-grained filtering material for sewage works, but the Committee feels there is still additional basic data badly needed by designing engineers. It is evident from these data, and from experience in existing plants that a good grade of silica sand, free from loam and other foreign substances, constitutes the best filtering medium for sewage sand filters. It is also known that sands of certain grading have handled successfully certain loadings. However, the exact tolerance that can be used in selecting filter sand, and the possible loadings for the different gradings, are not very well defined in existing data. Designers usually find themselves confronted with the problem of selecting sand from one or two and sometimes several sources, and if more definite data were available, the selection could be made on the basis of the sand offering the best economical possibilities. That is, with more definite data on possible loading with different gradings, a properly designed plant can be produced at a minimum cost. It is possible that seemingly identical sands as measured by the usual accepted standards of grading may differ materially in their efficiencies in sewage filters, due to different capillarities.

A study of the mass data already collected by the Committee disclosed the fact that variation of the relation between size of beds to the dosage is quite marked. It is also quite evident, that the data desired cannot be obtained from existing plants without extensive studies at each of a large number, due to the exceedingly large number of variables, such as grading of sands, character of applied effluent, operating personnel, etc. The Committee feels

that it will be necessary to obtain experimental data from one or more testing stations where tests can be run using different sands, different qualities of sewage, etc., and where each test run can be made with the variables reduced to the one to be studied. Therefore, an effort has been made to find where one or more such testing stations can be made available for this investigation, and where the investigation can be carried on under the general supervision of the Committee. While nothing definite has been determined as yet, several attractive possibilities have been found, and the next definite step in this phase of the work will be to get such investigations under way.

Considerable interest has been shown in the question of proper material for use in sludge beds. With the increased number of relatively large sewage treatment works being constructed, this phase of the subject of fine-grained materials appears to be relatively important and should warrant some definite study.

APPENDIX IV

Experiments to Determine Depth and Size of Sand for Water Filters

Detailed description of the proposed experimental work, including the operation of small glass filters was given in Appendix V of the Progress Report for 1927*. During the past year (1928) the battery of test filters required for the program of investigation has been installed by thirteen different co-operating cities.† The following is a list of these cities with the name and address of the engineer in charge of the filtration work:

Laboratories Co-operating in the Soundness Test of Trickling Filter Materials

(1) Kansas City Testing Laboratory, 700 Baltimore Avenue, Kansas City, Mo.
(2) National Crushed Stone Association Testing Laboratory, A. T. Goldbeck, Assoc. M. Am. Soc. C. E., Director, Merchandise Building, 14th and S Streets, N. W., Washington, D. C.
(3) Bureau of Standards Testing Laboratory, George K. Burgess, Director, Washington, D. C.
(4) Mr. Fred Hubbard, Bessemer Limestone and Cement Company Laboratory, Bessemer, Pa.
(5) C. C. Hommon, Assoc. M. Am. Soc. C. E., Chemist-in-Charge, Sewage Treatment Plant, City Hall, Canton, Ohio.
(6) Mr. A. S. Rea, Chief Engineer, Bureau of Tests, Department of Highways, Columbus, Ohio.
(7) Dr. Herbert F. Kriege, The France Stone Company Laboratories, 816 Summit Street, Toledo, Ohio.
(8) Pittsburgh Testing Laboratory, Stevenson and Locust Streets, Pittsburgh, Pa., Attention: Mr. J. W. Reifsnyder, Engineer of Tests.
(9) Penniman and Browne, Analytical and Consulting Chemists, 341 Courtland Street, Baltimore, Md.
(10) C. H. Scholer, Assoc. M. Am. Soc. C. E., Engineer of Tests, Road Materials Laboratory, Kansas State College, Manhattan, Kans.

* *Proceedings*, Am. Soc. C. E., April, 1928, Society Affairs, p. 272.
† *Loc. cit.*, p. 246.

(11) Gulick-Henderson Company, 431 South Dearborn Street, Chicago, Ill., Attention: Mr. E. J. Schwantes.
(12) Professor R. B. Crepps, Testing Materials Laboratory, Purdue University, Lafayette, Ind.
(13) George B. Gascoigne, M. Am. Soc. C. E., 1149 Leader-News Building, Cleveland, Ohio.

Sand for each of the batteries of test filters has been furnished by the Committee under the direction of Mr. James W. Armstrong, being shipped from Baltimore, Md., from the same source and screened by the same set of sieves.

Arrangements have been made to obtain routine reports from the several co-operating cities, following a standard plan of recording the results of the experiments.

A number of co-operating cities have shown considerable interest in the work, but in some places the work has dragged and the Committee has not been able to secure the anticipated results. It is believed that in most cities, slowness of the work can be accounted for by the fact that the laboratory force in charge of plant control has about all it can handle in the way of regular routine work, and the additional work required to keep up the records entailed by the experimental plant has been more than it can handle. At this time the data are not sufficient to warrant the drawing of any conclusions, but it is hoped that before another year passes, sufficient information will be in hand to enable the Committee to make some definite recommendations.

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FILTERING MATERIALS FOR WATER AND SEWAGE WORKS

PROGRESS REPORT OF THE COMMITTEE OF THE SANITARY ENGINEERING DIVISION

Discussion*

BY MESSRS. HARRY N. JENKS AND ALMON L. FALES.

HARRY N. JENKS,† M. AM. SOC. C. E. (by letter).‡—It is quite generally true, as pointed out in the Committee report,§ that most of the fundamental studies of trickling filter behavior in operation have been concerned largely with the biological aspects of the problem. As far as direct engineering applications are concerned, the results of these tests have lost some of their value, inasmuch as they have not been related in a definite way to many of the detailed physical features of plant design. As a consequence, engineers are often obliged to apply reported results of tests with a generous admixture of their own judgment, so that their completed designs should, as far as possible, conform to what is spoken of as "good practice". Whether or not this so-called "good practice" is in reality "good" will often depend whether one measures plant performance in terms of the actual results obtained or by such criteria as economy, reliability, flexibility, and freedom from nuisance, all in addition to meeting a specified standard of quality of effluent.

Without doubt, one of several promising lines of advance, whereby the engineer may pass from the realm of "good practice" to that of "better practice", is an engineering study of the limiting sizes of material, both for trickling and intermittent sand filters. In this connection, it appears to the writer that some of the investigations proposed by the Committee might well be directed toward the utilization of finer sizes of filtering materials which

* This discussion (of the Progress Report of the Committee of the Sanitary Engineering Division on Filtering Materials for Water and Sewage Works, presented at the meeting of the Sanitary Engineering Division, New York, N. Y., January 17, 1929, and published in this number of *Proceedings*), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

† San. Engr.; Assoc. Prof. San. Eng., Iowa State Coll., Ames, Iowa.

‡ Received by the Secretary, January 17, 1929.

§ See p. 1800.

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§ See p. 1800.

are known to be more efficient from the biological viewpoint, but which, as filters are now designed, cannot be safely used on account of clogging difficulties. Furthermore, additional information is needed on the proper dosing cycle for bacteria beds containing filtering material of a given size.

It has been recognized that one of the most serious objections to the trickling filter, from the standpoint of operation, is the tendency toward clogging that manifests itself in a markedly aggravated form when treating certain types of industrial wastes. Viewing this situation broadly, there seems to be good reason to believe that some positive and readily controlled means of "de-clogging" a trickling filter should enable the designer to make use of finer materials for a given filter loading, or of higher rates for a material of a given size. With this in mind, the writer proposed the use of a "de-clogable" trickling filter and designed a practical-scale unit for testing as to biological sufficiency, during recent research work at Iowa State College. The design was based on current practice in washing rapid sand filters used in water purification. The technique of washing follows closely that which prevails in filter plant operation. During the intervals between washings, representing a period ranging from one to three weeks, the filter functions strictly as an ordinary bacteria bed. The results so far obtained have confirmed the assumption that under these conditions, finer filtering material, even with higher rates, may be used. In view of such results it appears desirable that further engineering studies be made with respect to this special phase of trickling filter design.

Referring to the relationship of size and depth of filtering materials to the dosing cycle, the question arises as to whether the dosing cycle as ordinarily provided is in reality the optimum cycle for the material used in the bed being dosed. Recent tests on the effect of the dosing cycle on the actual rates of passage of the sewage through experimental filters have shown unmistakably that detrimental variations in retention period within the filter bed will occur when the dosing cycle is too long for the size of the filtering material. On the other hand, for any pre-determined cycle, it was shown that variations in the grading of the filtering material had a marked effect on the retention period, the finer materials exerting a storage effect within the bed out of proportion to the size of the material.*

The possible significance of these fluctuating rates of flow inside the filter medium may be judged in the light of certain studies reported in 1926,† which disclosed a tendency for the purification of milk-wastes to be an inverse function of the concentration and of the square of the rate of filtration. Similar studies on domestic sewage are needed. The economic bearing on engineering design, of results obtained from such investigations, is particularly evident in respect to the size and disposition of the filtering material in the trickling filter. For ordinary sized rock, the variation in rate of filtration may be as much as 300% from minimum to maximum. When the rate greatly

* "Methods of Research in the Treatment of Industrial Wastes", Thesis by Frederick G. Nelson, Iowa State Coll., 1928.

† "The Purification of Skimmilk Solutions on a Lath Filter", by Levine, Burke, and Linton, *Bulletin 81*, Iowa Eng. Experiment Station.

exceeds the average used in the design, the bed must necessarily operate at considerably impaired efficiency, if, as suggested by the studies cited, the purification is in inverse ratio to the square of the rate of passage through the filter. On the other hand, the minimum flows result in a corresponding failure fully to utilize the biologic capacity of the filtering medium. In general, it appears possible that the length of dosing cycles favored in practice might well be materially shortened.

ALMON L. FALES,* M. AM. SOC. C. E. (by letter).†—The Committee has undertaken investigations along three major lines:

1.—Size and depth of sand for water filters;
2.—Fine-grained material for sewage filter beds and sludge beds;
3.—Coarse material for sewage trickling filters.

The comparative tests with the experimental filters of different depths and sizes of sand, initiated by the Committee, at various water purification works, should yield information of much interest and value although, as stated in the report, these tests may have to be extended over a period of two years before definite conclusions can be reached.

The report states‡ that according to the information obtained, very few sand filters for sewage treatment are being constructed at the present time, but that the increased use of sludge beds warrants definite study of fine-grained materials for this purpose. It is true that sand filters are no longer being built for the treatment of the sewage of sizeable cities, but such filters are well adapted for many small communities where suitable sand is readily available. The importance of having a uniform grade of sand practically free from loam, clay, silt, and organic matter, has not been sufficiently well recognized. Satisfactory methods have been developed for the determination of organic matter in sand, sizes of grains, effective size, and coefficient of uniformity.

The report of the Committee deals largely with the development of laboratory tests for trickling filter material. In considering such tests, it is important to bear in mind the difficulties which have been experienced with trickling filter media and to adopt specifications and tests which will avoid these difficulties.

One of the prime considerations is the proper kind of filtering medium as respects the nature of its surfaces. The rough materials afford greater surface areas than the smooth materials and, consequently, are more efficient; but they tend to retard the unloading of accumulated solids and are, therefore, more liable to become clogged. Hence, it is desirable under ordinary conditions to avoid extremely rough materials like some slags or extremely smooth materials like gravel. The Committee has not yet taken up this phase of the problem.

The character of the material as respects soundness or liability to disintegrate is also an important consideration. Coke, cinders, and some rocks and slags are undesirable on this account. The investigations of the Committee have led to the conclusion that the most indicative test for soundness is the

* Cons. Engr. (Metcalf & Eddy), Boston, Mass.
† Received by the Secretary, January 17, 1929.
‡ See p. 1802.

sodium sulfate test. A freezing and thawing test would appear to simulate actual conditions more closely. However, the writer has found that results obtained by the sodium sulfate test approximate those obtained by freezing and thawing. The sodium sulfate test is really a physical expansion test and not a chemical test. Tests made by the writer indicate that it is important to carry out twenty cycles of this test, as indicated by the investigations of the Committee.

Materials having cementitious properties are to be guarded against. Some slags have this property, particularly when fine material is present. By weathering the slag for two or three years before crushing and screening and then very carefully excluding all fine material, the danger of amalgamation can be practically eliminated.

Hardness and toughness are important properties. Materials subject to breaking up on handling are undesirable. Even if re-screening at the point of delivery be practised, the placing of the filtering material and subsequent work on the surface of the filter may cause disintegration if the material is soft or brittle. The firm of which the writer is a member has included requirements with respect to hardness, toughness, and wear, in addition to the sodium sulfate test for soundness. Satisfactory methods have been developed for these tests.

Porosity is generally an undesirable property, but the possibility of trouble from this source is revealed by the soundness test.

The size and shape of the filtering material are important matters. Within limits, the finer the material the more efficient it will be as a purifying medium, but the lower will be its capacity and the greater the danger of clogging. The material should be as uniform as practicable, as pointed out by the Committee.* It is exceedingly important to keep out fine material which tends to fill the voids and promote clogging. If the fine material is confined to dust and very small particles, it may wash through the filter, but a graded material finer than the desirable size will not wash through. The sizes can be controlled by sieve tests. Materials which break into flat or elongated pieces should be excluded.

The importance of acceptance tests at the source cannot be over-emphasized. Specifications prepared by the writer's firm include the following:

"No filtering material shall be shipped by the Contractor until the source has been approved by the Engineer and such approval shall be subject to cancellation by the Engineer if the filtering material does not continue to meet the specifications. A period of at least thirty (30) days must be allowed for making inspection and tests prior to acceptance or rejection of the source."

It is much easier to prevent the shipment of unsuitable material than to exclude it on delivery. The sodium sulfate test which requires twenty days is obviously an impractical control test at the delivery end. Other tests can be made more readily at the contractor's plant. At Akron, Ohio, one or two inspectors were placed at the quarry to make the necessary observations and tests during the production of the filtering material.

* See p. 1801.

For the various tests it is, of course, of great importance that the samples be collected in such a way as to be truly representative.

The amount of work which might be done by the Committee in studying different phases of filtering materials both for water-works and for sewage works is almost unlimited. It will probably be more practicable for the Committee to determine the causes of difficulties experienced with filtering materials and to establish methods of avoiding such difficulties rather than to attempt to standardize practice with respect to the use of filtering materials. Local conditions, such as the character of the raw water or sewage, the extent of purification required, and the degree of preliminary treatment provided, will have an important influence on the filtering material selected.

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CHARACTERISTICS OF CENTRIFUGAL OIL PUMP

Discussion*

By Michael D. Aisenstein, Esq.†

Michael D. Aisenstein,‡ Esq. (by letter).§—The writer agrees with Professor O'Brien's desire to use the Reynold's law of similarity, which requires that the quantity, $\frac{v\,d}{\nu}$, be constant for dynamic similarity of flow;‖ but several factors that make the application of Reynold's criterion rather difficult should not be overlooked.

Argument No. 1.—Consider the various components of the total head:

$$H = h_w + h_{fw} + h_{sw} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (38)$$

in which,

H = total theoretical head for finite number of vanes.
h = total head.
h_f = friction head.
h_s = loss in head due to sudden enlargement.

The subscripts, w, and o, denote water and oil, respectively.

If the flow throughout the pump is turbulent, the relative nature of the liquids pumped is immaterial theoretically and:

$$h_w + h_{fw} + h_{sw} = h_o + h_{fo} + h_{so} \ldots\ldots\ldots\ldots\ldots\ldots (39)$$

Assuming for simplicity that the "shock loss" is not affected by viscosity,

$$h_{sw} = h_{so}$$

and,

$$h_o = h_w + h_{fw} - h_{fo} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (40)$$

In general, loss in head due to friction for a steady flow in straight circular pipes is given by the formula:

$$h_f = K\,L\,d^{n-3}\,v^n\,\nu^{2-n} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (41)$$

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* Discussion of the paper by Michael D. Aisenstein, Esq., continued from March, 1929, *Proceedings*.

† Author's closure.

‡ Chf. Engr., Pump Div., United Iron Works, Oakland, Calif.

§ Received by the Secretary, June 11, 1929.

‖ *Proceedings*, Am. Soc. C. E., March, 1929, Papers and Discussions, p. 811.

and for two different liquids when the flow is turbulent,

$$\frac{h_{fw}}{h_{fo}} = \frac{v_w^{\,n}\,\nu_w^{\,2-n}}{v_o^{\,n}\,\nu_o^{\,2-n}}$$

Since v is a function of G:

$$\frac{h_{fw}}{h_{fo}} = \frac{G_w^{\,n}\,\nu_w^{\,2-n}}{G_o^{\,n}\,\nu_o^{\,2-n}} \quad \ldots\ldots (42)$$

Substituting the value of h_{fo} from Equation (42) in Equation (40):

$$h_0 = h_w + h_{fw}\left(1 - \frac{G_o^{\,n}\,\nu_o^{\,2-n}}{G_w^{\,n}\,\nu_w^{\,2-n}}\right) \quad \ldots\ldots (43)$$

Experiments* indicate that for a curved rotating channel, the critical velocity is not proportional to the value of the coefficient of viscosity; that is, its value does not conform to the theoretical condition that $\frac{v\,d}{\nu}$ is constant.

Moreover, to solve Equation (43), it is necessary to know the laws of frictional resistance for revolving channels in liquids of different viscosity and the effect of viscosity on sudden enlargement losses. As far as the writer knows, this information is not available at present.

Argument No. 2.—In the model of a pump, the dimensions of which are truly proportional to the original, G is proportional to $u\ b\ d$, when u is the peripheral velocity, d is the diameter of the impeller, and b is the width of the impeller.

Since, when N equals revolutions per minute,

$$u = \frac{\pi\,d\,N}{60} \quad \ldots\ldots (44)$$

and b is proportional to $N\ b\ d^2$. In the same manner, h is proportional to $N^2\ d^2$.

For two geometrically similar pumps:

$$\frac{G_1}{G_2} = \frac{N_1\,d_1^{\,2}\,b_1}{N_2\,d_2^{\,2}\,b_2} \quad \ldots\ldots (45)$$

and,

$$\frac{h_1}{h_2} = \frac{N_1^{\,2}\,d_1^{\,2}}{N_2\,d_2} \quad \ldots\ldots (46)$$

If the ratio of all the linear dimensions between the given pumps is M, then from Equations (45) and (46):

$$G_1 = G_2\,M^3\,\frac{N_1}{N_2} \quad \ldots\ldots (47)$$

$$h_1 = h_2\,M^2\,\frac{N_1^{\,2}}{N_2^{\,2}} \quad \ldots\ldots (48)$$

and (correspondingly),

$$B\,H\,P_1 = B\,H\,P_2\,M^5\,\frac{N_1^{\,3}}{N_2^{\,3}} \quad \ldots\ldots (49)$$

These equations based on dynamic similarity are found to be approximately correct.

* Staunton, "Friction".

For instance with ratios of 1 : 2, the specific speed of pumps changed 7%, although theoretically to satisfy the dynamic similarity the specific speed should have been constant.

The viscosities of oils being moved by centrifugal pumps reach a value of 6 000 S. s. u. (Saybolt seconds universal) for larger pumps and as much as 3 000 S. s. u. are very common.

With corresponding ratio of kinematic viscosities of approximately,

$$\frac{\nu_0}{\nu_w} = 1\ 295 \text{ and } 648$$

For $\frac{\nu_0}{\nu_w} = 18$, which was given by Professor O'Brien* as most common in practice, the head capacity is practically the same for oil and water for pumps 3 in. in size and greater. The only change which takes place is in horse-power which increases slightly due to increased disk friction depending on the diameter of the impeller and the operating speed. Consequently, little new information would be gained by testing pumps within the range of viscosities mentioned by Professor O'Brien.

For higher values of viscosities the method of testing pumps suggested by Professor O'Brien does not seem to be practicable, because of the "scale effect" and due to reasons outlined in Argument No. 1.

Professor O'Brien asks* how the oil-pumping characteristics predicted from water characteristics given in the paper compare with the actual oil-pumping characteristics, as obtained by experiment. The agreement with the actual tests was very close.

* *Proceedings*, Am. Soc. C. E., March, 1929, Papers and Discussions, p. 813.

For instance with ratios of 1 : 2, the specific speed of pumps changed 7%, although theoretically to satisfy the dynamic similarity the specific speed should have been constant.

The viscosities of oils being moved by centrifugal pumps reach a value of 6 000 S. s. u. (Saybolt seconds universal) for larger pumps and as much as 2 000 S. s. u. are very common.

With corresponding ratio of kinematic viscosities of approximately,

$$\frac{\nu_1}{\nu_2} = 1.500 \text{ and } 1.8$$

For $\frac{\nu_1}{\nu_2} = 1.8$, which was given by Professor O'Brien* as most common in practice, the head capacity is practically the same for oil and water for pumps 1 in. in size and greater. The only change which takes place is in horse-power which increases slightly due to increased disk friction, depending on the diameter of the impeller and the operating speed. Consequently, little new information would be gained by testing pumps within the range of viscosities mentioned by Professor O'Brien.

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* Proceedings, Am. Soc. C. E., March, 1929, Papers and Discussion, p. 875.

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THE RELATION BETWEEN EARTHQUAKES AND ENGINEERING SUBSTRUCTURES

Discussion*

BY HENRY D. DEWELL, M. AM. SOC. C. E.†

HENRY D. DEWELL,‡ M. AM. SOC. C. E. (by letter).§—Nearly three years have elapsed since this paper was written, and three years of continued study of any subject should result in a clearer and broader knowledge of both its possibilities and limitations. This statement is particularly true of an engineering subject, the development of which is yet in its infancy. The studies incident to the preparation of the final report of the Special Committee on the Effect of Earthquakes on Engineering Structures have clarified many points not before understood.‖ Particularly they have brought out the limitations of present knowledge of the fundamental characteristics of earthquake waves in the region of destructive action. The present ability of structural engineers to design safely against earthquakes is definitely limited by the absence of accurate and complete seismograms of major earthquakes as might have been actually recorded at the points where engineering structures suffered serious damage or were destroyed.

The writer appreciates the interest that has been evidenced. Some of the discussions have exceeded the limitations of the subject, as stated in the title of the paper, namely, engineering substructures. These digressions are not to be criticized, inasmuch as substructures and superstructures are so intimately related, particularly with respect to resistance to earthquake, that separate treatment is very difficult, and, at times, impossible. Indeed the writer was not guiltless in this respect; he found it difficult to keep within the limitations of his subject.

* Discussion of the paper by Henry D. Dewell, M. Am. Soc. C. E., continued from August, 1929, *Proceedings*.

† Author's closure.

‡ Cons. Engr., San Francisco, Calif.

§ Received by the Secretary, April 30, 1929.

‖ This report has not been published as yet.

Mr. Spear has clearly stated* the requirements for safety from earthquake damage in a large city; that is, safety of buildings from collapse, either in part or in whole, means of transportation, and continuity of water service. Falling masonry, as of heavy cornices is a real menace. Nature has seen fit to be somewhat considerate in the time of staging her California convulsions. The great shock of 1906 occurred at 5:18 A. M.; the rather minor Inglewood and San Jacinto shocks came after business hours and on Sunday afternoon, respectively; and the Santa Barbara earthquake of 1925 was in the early morning hours. Even the great Charleston earthquake occurred at night. The difference in the probable loss of life due to a major earthquake occurring when business houses and schools are full, as compared to a time when most of the populace are in their homes, is great. The well-built dwelling house is a comparatively safe place in a destructive earthquake.

Mr. Spear is right in emphasizing the duty of the engineer to make his structures reasonably safe in all parts from earthquake shock. The great need, however, is for a general understanding and interest on the part of the public as to the necessity of taking reasonable precautions against such shock. When this interest and understanding is general, the engineer's task will be very much easier. It is exceedingly difficult to oppose successfully and continuously that public sentiment which desires to avoid discussion of possible earthquake damage.

On the other hand, it is very doubtful if the methods for informing the public on the subject should include that of attempting to frighten unnecessarily. Professor Jaggar's "remedy",† a recommendation for more seismographs, is not without reason. The present knowledge of the effect of a destructive earthquake on very high buildings is largely conjectural, and apparently must remain so, until designers have more definite knowledge of the earthquake wave, or until a destructive earthquake in a city of very high buildings provides an actual test.

Mr. O'Shaughnessy gives‡ pertinent information as to the effect of the California earthquake of 1906, particularly on the Crystal Springs Dam, the Upper Crystal Springs Dam, of the Spring Valley Water Company, near San Francisco, and as to the action of the Sheffield Dam, near Santa Barbara, in 1925. While it is true that the rock-fill dam at Morena, in San Diego County, California, is in a seismatically active region, the records indicate that this dam has never been subjected to an earthquake of greater intensity than VI of the Rossi-Forel scale; hence, its present stability should not be advanced as proof that a rock-fill dam is safe against earthquake shock. The Crystal Springs Dam is of unusually heavy construction, having been designed to be capable of a future increase in height, as is pointed out by Mr. Jakobsen.§

In speaking of the San Francisco earthquake of 1906, and the damage suffered therefrom by buildings, Mr. O'Shaughnessy makes statements‖ that,

* *Proceedings*, Am. Soc. C. E., December, 1928, Papers and Discussions, p. 2749.
† *Loc. cit.*, p. 2750.
‡ *Loc. cit.*, p. 2751.
§ *Loc. cit.*, January, 1929, Papers and Discussions, p. 226.
‖ *Loc. cit.*, December, 1928, Papers and Discussions, p. 2751.

if taken literally, are inaccurate. It is not correct that, in the 9½ acres mentioned by Mr. O'Shaughnessy as having been destroyed by conflagration, all but seven private buildings collapsed from the effect of the earthquake; neither is it correct that the U. S. Post Office Building and the Ferry Building escaped damage. Photographs of the principal business district, included in the 9½ acres mentioned, taken just after the earthquake and before the fire, clearly show relatively little damage, and little or no débris in the streets. Some parapets are missing, and some walls are visibly cracked, but collapsed buildings are conspicuous by their absence. On the other hand the walls of the U. S. Post Office and the Ferry Building, were severely cracked, and the tower of the Ferry Building was so badly damaged that it had to be taken down.

In a letter sent in reply to an inquiry from the writer, Mr. O'Shaughnessy states:

"I am particularly familiar with the Post Office Building situation because, when I returned from Hawaii, I went out Mission Street to that point and saw that the sidewalks on the natural ground surface of the whole street had settled 4 ft.

"The main structure of the Post Office Building is founded on piles and did not suffer substantial damage, and when I speak of damage I am dealing with a big measure and broad terms. The same may be said of the main Ferry Building. The tower in front of the Ferry Building was practically wrecked, due to defective framing and also due to its height. The main Ferry Building was substantially intact, because I passed through there as a commuter, going back and forth to Marin County, and have a vivid recollection of its condition."

Mr. O'Shaughnessy's recital of the provisions that have been made in the Hetch Hetchy Aqueduct to withstand future earthquakes is interesting, and shows that reasonable precautions against such shock are not difficult to make, and may be expected to be effective in greatly reducing the possibility of a repetition of the 1906 San Francisco conflagration.

Dr. Willis presents a scholarly discussion. His expressed reservations* to the writer's conclusion with respect to the safety of earthfill dams are properly in order.

Mr. Jakobsen presents† a somewhat detailed consideration of the probable effect of an earthquake on masonry dams, both of the gravity and the arched types. He also introduces the subject of uplift on foundations, due to water pressure, and the effect of arching gravity dams. As the discussion of these later subjects does not seem to be directly pertinent to the effect of earthquakes on dams, and is somewhat of a controversial nature, no comment will be made by the writer. It is pertinent to point out that Mr. Jakobsen considers only the case in which the direction of shock, or of vibrations, is normal to the longitudinal axis of the dam; that is, up stream and down stream. It has seemed to the writer that the action of the dam when subjected to shock or vibrations in a direction parallel to its axis, or across the stream bed, might be expected to induce much heavier stresses in an arched

* *Proceedings*, Am. Soc. C. E., December, 1928, Papers and Discussions, p. 2753.

† *Loc. cit.*, January, 1929, Papers and Discussions, p. 219.

dam than in the first case. Such a comment would appear to be particularly pertinent to a dam of the multiple-arched type, in which use is made of high slender buttresses, but lightly braced.

Mr. Noetzli brings forward* the cases of the Gibraltar Dam which successfully withstood the Santa Barbara earthquake, and the Carfino Arch Dam, in Italy, as examples of the earthquake resistance of arch dams. The writer might add to the list of masonry dams which have passed through heavy earthquakes, the Williams Dam, near San José, Calif., a concrete dam of gravity section, not arched, and about 80 ft. high, which was uninjured in the 1906 shock, and the Hemet Dam, in California, a massive gravity dam arched in plan, which passed through the San Jacinto earthquake of April, 1918, without damage. At the time of the earthquake, this dam was 121 ft. high, about 100 ft. thick at the base, and approximately 25 ft. thick at the top. Later, the height was increased.

With respect to the Lake Hodges Dam, a multiple-arch structure, near San Diego, noted by Mr. Noetzli as having been subjected to "a fairly severe earthquake", the writer's comment previously made with respect to the Morena Dam is applicable. Neither dam has had, as yet, the test of an earthquake of destructive violence.

In reply to the charge of Mr. Floris† that the paper contains considerable material that has been published elsewhere; even in articles by the writer which are classed by Mr. Floris as "numerous," the writer pleads "not guilty" of having submitted his paper for publication in *Proceedings*. The decision to publish the paper was made by others.

Mr. Floris states very confidently that (1) in the case of earthquakes with origin from 6 to 30 km. in a vertical direction from a seismograph, "the motion imparted to the seismograph begins with very nearly, if not quite, its full amplitude"; (2) "both kinds of waves [longitudinal and transverse] appear almost simultaneously"; and (3) that the part of the "earthquake that is disastrous to structures often produces the same effect as impact, and its energy dies out quickly after a few oscillations. Structures in earthquake areas must be able to withstand such strong shocks." Obviously, Mr. Floris refers to shocks of destructive intensity. These statements appear reasonable, but the writer knows of no seismograms of destructive earthquakes made by an instrument the record of which is accepted by seismologists, that show what Mr. Floris states to be true. Mr. Floris uses the terms "impact" and "impulsive forces" in speaking of these forces, yet in the next sentence says "only the horizontal component is of importance in structures, and that may be assumed to act statically."

The writer agrees that there is good reason to believe that a structure so situated as to be relatively near to the focus of a destructive earthquake will be as Mr. Floris states, subject to an "impact effect". Testimony of some residents of Santa Barbara in 1925 was that a sudden "bump" or "blow" was the first effect felt of the earthquake; but, as stated previously, the writer has been unable to find any seismographic record that corroborates this belief;

* *Proceedings*, Am. Soc. C. E., March, 1929, Papers and Discussions, p. 817.

† *Loc. cit.*, August, 1929, Papers and Discussions, p. 1589.

in fact, he is informed by seismologists that no such records exist. The writer agrees with Mr. Floris that the horizontal force "can be easily determined, provided the acceleration of the earthquake movement is known". The last clause of the sentence, only, is of importance. When dependable measurements of the maximum of the many accelerations of a destructive earthquake are available, most, if not all, of the present limitations of knowledge of earthquake-resistant design will have disappeared.

Mr. Floris then briefly discusses possible resonance in the case of high slender structures, introducing this discussion by the remark, "There is also one important phenomenon that the author had scarcely mentioned." The writer might remark, in reply, that there are several phenomena which he could have, but did not, mention. The discussion of "high slender structures" was not considered as properly coming within the scope of a brief paper on engineering substructures.

With reference to the statement of Mr. Floris that "it is a well-known fact that earthquake waves in an alluvial soil are of great amplitude and very low speed", one must remember that alluvial territory, particularly, may be subjected to two kinds of waves in an earthquake, namely, what may be termed as "elastic waves" and what may be called "surface waves", the latter not being registered on a seismograph. It is not clear to which of the two types of waves Mr. Floris refers.

Until many more data are available on the nature of the elastic waves of an earthquake of destructive intensity, and until dependable measurements have been made of the secondary ground movements, the writer believes that judgment of the probable action of inelastic structures in an earthquake should be governed by actual experience.

Mr. Kelch* perhaps places more confidence in the value of deep foundations than the writer. While there is much evidence to indicate that deep foundations are a definite asset to any structure for resistance to earthquakes, theoretical considerations would indicate that such advantage would be largely nullified by contact of the structure with the surface ground. Absence of such contact is inconvenient, and sometimes impossible. The writer knows of no reliable data by which the advantages of deep-seated foundations can be quantitively evaluated.

With reference to the values of interconnected footings, the writer would argue that purely theoretical considerations indicate the benefit of such interconnections, in that such ties and struts would tend to make more certain simultaneous equal movement of all parts of the foundations. Having in mind the complexity of earthquake movement, he is not certain that one can rely on all parts of the ground under a structure having simultaneous movement. On the other hand, it may be pointed out that the Special Committee of the San Francisco Association of Members of the Society, reporting on the effects of the San Francisco earthquake in 1906, stated,†

"Some discussion has taken place as to the advisability of making a monolithic mass under buildings. Several of these have been constructed, such as

* *Proceedings*, Am. Soc. C. E., August, 1929, Papers and Discussions, p. 1591.

† *Transactions*, Am. Soc. C. E., Vol. LIX (1907), p. 235.

the Claus Spreckels, Mutual Savings Bank, and Bullock and Jones Buildings. They are all of relatively small base. * * * Buildings of that size [12 stories high] and larger, with isolated pier foundations, suffered no more than others. The evidence is that foundations well built, along accepted lines, are adequate".

With respect to sewer pipe, the writer is in sympathy with the arguments of Mr. Kelch for providing as much flexibility for pipe lines as possible, from the standpoint of offering resistance to breakage by earthquake. Some sewer lines in Santa Barbara were broken* but of just what the damage consisted, was not known at that time. It is possible that vitrified pipe and reinforced concrete pipe will behave equally well. Opinions of engineers on this point, as on others, are bound to vary.

Summing up the subject of the effect of earthquakes on dams, which seems to have been of the most interest to discussors, the evidence available is very favorable to the masonry dams, both of the gravity and arched types. The fact that masonry dams are usually, if not always, supported on rock foundations, and are, therefore, in areas of lower acceleration than the soil on which earth-fill dams are constructed, is probably an important factor. Again, a dam is a very different type of structure from a building, in that it is in contact with the ground for its full height. In general, this characteristic would appear to be a favorable one. Dams having great length, however, might have their ends, near the crest, subjected simultaneously to shocks with directions not parallel, thus subjecting the structure to serious distortion.

The limits of the paper have not permitted a detailed discussion of the possible stresses in dams due to earthquake shock. The paper and discussions have indicated that, except where actual differential or permanent movement of the foundation takes place, dams of all types, if designed and constructed with due and reasonable regard to probable lateral forces, should safely withstand earthquake shocks.

* *Engineering News-Record*, July 23, 1925, p. 146.

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PAPERS AND DISCUSSIONS

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REPORT OF THE COMMITTEE OF THE IRRIGATION DIVISION ON "A NATIONAL RECLAMATION POLICY"

Discussion*

BY MESSRS. FRANK ADAMS, M. J. DOWD, C. E. GRUNSKY, S. T. HARDING, AND H. H. DARE.

FRANK ADAMS,† M. AM. SOC. C. E. (by letter).‡—The writer is in hearty agreement with most of the recommendations embodied in the report of the Committee.§ His remarks will be confined to those recommendations, or parts of recommendations, with which he does not fully agree.

Item (1)‖ in the report holds that the waiving of interest payments to land owners on Government reclamation projects is unwise and that in the future Government contributions should appear in the assumption of a part of the cost of project works.

In considering this recommendation one should have clearly in mind that the principal object of Federal reclamation has been to reclaim areas which without Government aid would have remained idle or unproductive. In some cases, it has been the large amount of capital required for the construction of reclamation works which put them beyond the means of private and district enterprise. Again, it has been the excessive unit cost, and the long time required to complete the reclamation through actual settlement and utilization of the land. In still other cases, it was the inability of private or district enterprises to place on the land reclaimed responsibility for paying the cost of the necessary reclamation works. In a few instances, most notably the pending Boulder Canyon development, the Federal Government has been the only agency politically competent to cope with the related interstate and inter-

* Discussion of the report of the Committee of the Irrigation Division on A National Reclamation Policy, continued from August, 1929, *Proceedings*.

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‡ Received by the Secretary, June 24, 1929.

§ *Proceedings*, Am. Soc. C. E., September, 1928, Papers and Discussions, p. 2097.

‖ *Loc. cit.*, p. 2098.

national complications. The Reclamation Act was passed, therefore, not primarily to bring more land into agricultural production, but as an aid to Western development, which, at the time the Act was passed, was lagging. It was very generally accepted as a desirable measure designed to put the opportunities for development in the arid and semi-arid West on a par with those in the East and Middle West.

In carrying out the National Reclamation Act, it was both desirable and inevitable that the Government should proceed along more comprehensive lines than would have been justified in the case of strictly commercial and district reclamation enterprises. This meant not only added cost in surveys and construction, but also the opening of lands for settlement on such a scale as to involve long delay in their full utilization. In fact, few of the Federal reclamation projects are yet completely settled. The burden of settlement, which has fallen almost wholly on individuals and inadequate local credit agencies, has been a heavy one. Nevertheless, the advantage to many Western States and communities has been tremendous, and the nation has benefited greatly through this process of equalizing opportunities for its people to create homes and develop industries in those parts of the country which, although rich in resources, still lay more or less backward and dormant.

No such reclamation program as has been or is being carried out under the National Reclamation Act would be possible without a large financial contribution by the Federal Government. This has always been recognized and the contribution has been made by waiving interest on the cost of reclamation construction. This procedure has been fully justifiable in view of the purposes to be accomplished. However, the waiving of interest has not been stressed, and, consequently, the actual amount of the Federal contribution has not been known. Therein perhaps lay the chief evil of the system.

There is no question that direct assumption by the Federal Government of that part of the cost of reclamation works which it is to pay, as recommended in the Committee report, is abstractly the more desirable procedure; but will it work? That is very questionable. The writer believes, therefore, that until Congress is ready to face the facts frankly and to adopt direct Government aid, whether in the West, East, North, or South, as conditions may warrant, the present plan of contributions by the waiving of interest should not be abandoned. The work of Federal reclamation has not yet been finished because there are still desirable projects pending which can not be built without Government assistance. The present plan has worked, even if not always entirely satisfactorily, and should be retained until some other plan has been shown to be feasible. The writer is ready to assist in bringing about a change in the public viewpoint, but he is not ready to condemn as unwise the plan that has been responsible for such great benefits as have been accomplished thus far under the National Reclamation Act.

There is another phase of the waiving of interest on the cost of Federal reclamation construction that should be mentioned, for it should be considered when and if the plan of direct Federal assumption of part of the cost, as recommended by the Committee, is put into effect; that is, the need for waiving interest during the period in which settlers on the projects are becoming

established. The period during which land is being cleared, leveled, and checked for irrigation, and brought into production, is that in which settlers most need help, but find it hardest to obtain. The land is not then earning either interest or operating expenses. Lacking help, many of the settlers fail, not only on Federal, but also on commercial and district projects. The Government could well afford to forego interest on its projects during this period, regardless of whether interest is charged later.

Item (6)* of the Committee recommendations holds that the output of power and of water at Federal or State works should be disposed of at wholesale and not at retail. If the Committee would insert the phrase, "in general", the writer could agree. Many public agencies, some of them irrigation districts, have successfully retailed both water and power and are still doing it. It is conceivable that in some cases both the Government and the States will find it desirable to retail the output, perhaps only temporarily, and a policy should not be so crystallized against such procedure as to preclude it when necessary to accomplish the largest benefit, whether to the Government, to the State, or to the consumers. Each case should be decided on its merits and not in advance by doctrine.

Committee Recommendation No. (9)* that, except in the case of commitments already made, it is undesirable for the Federal Government to bring new areas under cultivation, coincides with recommendations made by numerous farm organizations, as well as by some of the agricultural economics experts of the Federal Agricultural Department.

The writer does not believe that over-production in certain branches of agriculture and in certain areas of the country at any particular time is necessarily any reason for ceasing the reclamation of new areas if the projects which would reclaim those new areas are themselves economically feasible, and desirable because of their contribution to the general welfare of tributary areas. There may be over-development in the country generally at the same time that there is under-development in certain important areas. The test in beginning a new project should be: Is it economically feasible, when viewed in the light of long-time production and price trends, and is it in line with equalizing opportunity as between different sections of the country? Furthermore, can it be settled within a reasonable time? If the answer to either of these questions is in the negative, construction of the project should be postponed.

The Committee probably was led to recommend that no new projects be started at this time by the lack of needed settlers on some of the existing Federal projects and in many irrigation districts. Admittedly, the demand for land is not active at this time, partly because general agricultural conditions have been unsatisfactory in some parts of the country, and partly because of the small number of settlers with the necessary capital and farm experience to make a success under the generally high costs of present-day projects. However, there is need for Federal aid in some States in connection with supplementing the water supplies of existing enterprises. To cite

* *Proceedings*, Am. Soc. C. E., September, 1928, Papers and Discussions, p. 2099.

a single example, California is studying her water resources with a view to the adoption of a comprehensive plan of water utilization. There are areas of surplus water and areas of deficiency, and the cost of equalization will be very high. The benefits to accrue will ultimately justify the expenditure, but there is need of Federal aid in financing, as well as in planning and building the necessary works. Other States have similar problems, and it will require many years for their solving. In the meantime, economic conditions will continue to change and much of the present slack will be taken up.

The writer also desires to take exception to Item (14)* in the Committee's recommendations. The first sentence reads:

"Land settlement, including paternalistic, financial, or any other kind of assistance to the individual farmer, should be treated as a local matter and should therefore be made the concern of the State or locality rather than of the United States."

While firmly believing in the soundness of Governmental land settlement policies that have been successfully put into effect in some other countries, the writer concedes that the present public attitude is generally against those policies for this country. However, if the Government is merely to make existing reclamation projects successful it must do what is necessary to complete their settlement as rapidly as possible. The Government is not justified in asking settlers to go on its projects unless conditions are right for their success.

The Government should at least put the land in the projects into such condition that the new settlers can get a foothold without exhausting their meager capital and being forced to abandon their farms before they are on a profitable basis. The expenditure of $20 to $40, or more, per acre for leveling land and otherwise preparing it for irrigation is not unusual on some of the desert-land reclamation projects. This work could be done far more economically by the Government, working on a large scale with heavy machinery, than if left to the individual farmers. The writer does not advocate having the Government prepare for irrigation all the land on its reclamation projects, but he believes that when the cost of preparation is high and the prospective income per acre is relatively low, both the Government and the individual settlers would be the gainers if the preparation of enough of each farm to permit the immediate planting of perhaps one-half of it, or a minimum of 20 acres, were to be included in the Government reclamation program, the cost of this to be paid, with interest, within a relatively short period after settlement.

This would foster settlement and increase the percentage of individual success by advancing the time when the farms would be producing an income. Failure to obtain a quick income, coupled with the heavy expense of early farm development, and the general lack of local credit, except to those who have substantial resources, is the chief cause of failure of both settlers and projects. Theoretically, the State or some local agency might do this preliminary work which is of so much importance to the settler. Practically, however, it is never done, nor is it likely to be undertaken except by the

* *Proceedings*, Am. Soc. C. E., September, 1928, Papers and Discussions, p. 2100.

agency which is most vitally concerned, which, in the case of Government reclamation projects, is the Government itself.

There is still another function which the Government should perform on its reclamation projects but which the Committee might class as "paternalistic". This is organizing the community development and advising as to cultural practices and types of production, the latter being perhaps of most importance when the settler is just starting. Possibly the Governmental agency that builds reclamation works is not the one that is best fitted to lead and advise in such matters, although it is not clear that this is the case. At any rate, some Governmental agency should be charged with that responsibility, because the Government should not let go as long as its funds are still tied up. The help of local agencies should of course be obtained to the limit, and in some cases such agencies could carry the main part of the burden; but so long as the project is under Federal control, the Government should be privileged to perform such functions as deemed necessary by those who carry the responsibility for its success. Otherwise, the Government should stay out of reclamation and confine itself to constructing major works, such as the Boulder Canyon, or some of the other large dams built by the Bureau of Reclamation. Real reclamation is not accomplished until the settlers on the project lands are well on their way toward being established.

It is exceedingly desirable that all the worthy Federal reclamation projects should reach the successful stage at the earliest possible date. The Government can afford the loss that will accrue to the reclamation fund through delay, but the settlers on the projects cannot, because such projects are now very largely organized as irrigation districts in which all land owners have a joint liability for the construction charges and costs of operation and maintenance. Any competent and solvent business man would supply whatever aid is necessary to complete the success of these projects that can succeed, and there is no justification for the Government not doing as much because of a fear of being "paternalistic".

M. J. Dowd,* Assoc. M. Am. Soc. C. E. (by letter).†—It is apparent that the Committee has given considerable thought and study in formulating the principles of "A National Reclamation Policy". The writer is in accord with a majority of these principles, but would like to comment on one or two of them which in his opinion should be questioned.

In considering Principle (1)‡ it should be remembered that the reclamation fund is built up from sales of public land and not from taxation. The arid public land is of little value until water and labor are applied. If the original capital costs the Government nothing and it is to the interest of the country that the non-productive valueless land be made productive and if an individual can be found who will set his labor against the free capital which the Government furnishes, it does not seem as if the Government would be justified in making a profit out of the transaction in the way of interest or otherwise. Certainly all the money which comes out of

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† Received by the Secretary, July 15, 1929.

‡ *Proceedings*, Am. Soc. C. E., September, 1928, Papers and Discussions, p. 2098.

the sale of public land should go back into the development of such land and in so doing the whole Nation profits by the increased wealth. Even where the Reclamation Fund is used to develop land in private ownership, which was formerly public land, the writer believes such land should not pay interest on the capital invested; but in any event, where the development concerns public land alone it seems as if the present policy of not charging interest certainly should apply.

The enforcement of Principle (9)* would mean placing an embargo upon agricultural development in the West which is both unwise and unnecessary. The following quotation is taken from a speech of President Coolidge, regarding reclamation in the West:

"A country growing so fast as is our own cannot limit its considerations to immediate necessities. Many people now living will see this a country of two hundred million inhabitants. The national interest will be served by promoting the wisest diffusion of population. The national security, the best social development, the wisest untilization of natural resources, all demand this. Criticism of such a program of encouragement, on the ground that there is already an over-production in agricultural products, lies in the lack of understanding that the projects take many years for development; that they furnish only a small portion of the total increased food supply required even by our increasing population; and that the utilization of their supplies lies in the development of the West itself. It is my purpose, unremittingly, to stimulate and encourage the development of these great projects by every authority of the Federal Government."

The writer believes it is a mistaken idea, as expressed in Principle (9), that reclamation should be curtailed simply because, for the moment, there is an over-production in this country of certain staple commodities. Reclamation projects that are easy of development, have been completed and those that are now being considered are not only complicated but costly, which means that future argicultural development in the West necessarily depends upon reclamation by public agencies.

A reclamation project requires from 5 to 25 years to develop. There are only about 20 000 000 acres of unreclaimed arid land for which there is a water supply available or for which such a supply can be made available. If it were all put to crop as rapidly as possible, it would not be even a serious factor in the National agricultural requirements by 1950. It is estimated that the population then will be at least 150 000 000, requiring production from about 431 000 000 acres in addition to pasture lands. In 1919, there was a total of about 365 000 000 acres in crops harvested. In other words, by 1950 the country will require about 66 000 000 acres more land for crop purposes, with the same yield per acre and the same consumption per person. If all the 20 000 000 remaining irrigable acres in the Western States are put under irrigation, there will yet remain a shortage of 46 000 000 acres 21 years hence. This indicates that in order simply to keep pace with normal development there should be reclaimed about 1 000 000 acres per year.

During the period of 1910-20 the population of the country increased about 15% and in the western part, nearly 35 per cent. These Western

* *Proceedings*, Am. Soc. C. E., September, 1928, Papers and Discussions, p. 2099.

States, and particularly California, import a great quantity of their food supply at the present time. California exports virtually nothing in quantity except fresh fruits and vegetables. The coastal cities import from without the State from 10 to 40% of their dairy products.

It is safe to assume that the population of the Western States will increase faster in the future than it has in the past, because of numerous economic conditions, such as climate and the Pacific trade. Therefore, a program of retarded reclamation simply means increased cost of food supplies for the West and a consequent slowing up of western development.

As has already been stated, the carrying out of the policy proposed by this principle means an agricultural embargo on the Western States, leaving the more humid sections of the country to develop as rapidly and as much as they may desire. With approximately 40% of the total area of the United States within the arid States, the placing of such an embargo on agriculture for the benefit of the remaining 60% is gross injustice. Furthermore, the adoption of this principle in no manner affects reclamation of additional areas in either Canada or Mexico.

In the case of Mexico, there has been in recent years a rapid development of reclamation near the American border and many hundreds of carloads of peas, tomatoes, lettuce, etc., are coming into the United States every year. Because of economic conditions in Mexico such products can be produced cheaper than in this country. With proper proctection against competition from this source, there is at present the market for a considerable increase in the vegetable industry in the United States; but without such protection, even those projects now producing fruits and vegetables will not long be able to compete with the foreign growers. Such protection is economically sound and in accord with the tariff policy of this country.

There is no question as to there being an over-production in certain staple commodities produced in the United States, but this does not hold true for all products. In the Western States, at least in California, there is a shortage of dairy products, hogs, and beef cattle. It has been noted that what often appears to be over-production is simply poor distribution. This applies particularly to the vegetable and fruit industry, where it has been shown many times that the returns to the grower are governed primarily by the manner in which the product is distributed rather than by the supply of the product. Furthermore, there should be considered the rapid increase in the use of certain farm products, such as fruits and vegetables. The saturation point for the orange production was predicted many years ago and yet, because of increased use and co-operative marketing, the returns to the grower are better to-day (1929) than they were ten years ago. As a further illustration, consider the lettuce industry in the Imperial Valley. Shipment from 2 000 acres of lettuce in 1913 flooded the market and the prices received would hardly pay the freight; yet in 1929 about 12 000 carloads were shipped and the net profits are estimated at from $2 500 000 to $3 000 000. The same holds true for the cantaloupe industry in Imperial Valley. In 1908, the market could not profitably absorb 1 804 carloads of

cantaloupes but, in 1927, more than 17 000 carloads were shipped from Imperial Valley alone and very good returns were received by the growers.

Indications all point to the end of the agricultural depression which followed the World War. Because of this, and the fact that during the intervening period, there has been only a slight expansion of reclamation projects, the writer believes the time has arrived for a resumption of the development of Western arid lands.

C. E. GRUNSKY,* PAST-PRESIDENT, AM. SOC. C. E. (by letter).†—The report‡ of the Committee on a National Reclamation Policy is necessarily more or less of a compromise. Members of the Committee without retracting in any measure the views expressed in the report remain free to amplify their attitude toward the policy therein outlined and will, it is hoped, contribute discussions which may throw much needed light on a problem that has vexed other countries besides the United States.

In line with this thought the writer will refer to several points which should receive somewhat more attention than has been given them in the supporting statement§ of Chairman Lippincott.

In the first place it seems fundamentally wrong and unwise to make indirect appropriations for the carrying out of public improvements, such as land reclamation. If a project is worthy and deserving of Government aid the appropriation of funds therefor should be a direct appropriation for a specified purpose. Under the Reclamation Act the moneys received from land sales and, in part, from oil royalties drift into a fund which, for a time, was at the disposal of the Secretary of the Interior; since 1914, however, the Secretary's allotments are subject to authorization by Congress. Losing sight of the fact that such moneys would naturally have gone into the United States Treasury and have been there at the disposal of Congress, it was prescribed in the Reclamation Act of 1902 that these moneys should be used by the Secretary of the Interior for the reclamation of arid or semi-arid lands. One-half the money collected in any State should be expended within that State if there were available projects. Due to political exigency, an unnatural demand for the expenditure of funds on work of doubtful merit resulted and this fact may account for the undertaking of some of the earlier projects which should have been let alone. Believing that the proceedings under the law were unsound in principle, unwise, and not in the best interest of the development of the country's natural resources, the writer is opposed to the indirect appropriation and believes that, in this respect, a new policy should be adopted.

In the second place the aid extended by the Federal Government when wet or arid lands are to be reclaimed should not go beyond the regulation of stream flow and the control of floods. The water of rivers and lakes together with underground water is this country's most valuable natural resource. It has been the practice to allow the title to reservoir lands to be acquired by private parties, by districts, and by municipalities without regard to the con-

* Cons. Engr. (C. E. Grunsky Co.), San Francisco, Calif.
† Received by the Secretary, June 17, 1929.
‡ *Proceedings*, Am. Soc. C. E., September, 1928, Papers and Discussions, p. 2097.
§ *Loc. cit.*, May, 1929, Papers and Discussions, p. 1193.

struction of works in conformity with a comprehensive plan and without Government control of the manipulation of the reservoirs. The regulation of the stream by storage should be such as to produce "the greatest good to the greatest number." Consequently, the people should never allow works for the regulation of the flow of a river to get out of Government control.

This does not mean, however, that the United States or the individual State should be the only agency for the actual construction of works. The privilege should be granted to any State, to a group of States, to municipalities, to districts, and even to private parties, always provided, however, that the works conform to a comprehensive scheme of development and that permits shall be conditioned upon use of the storage facilities in conformity with the regulations prescribed in the case of the international, interstate, or navigable streams, by the United States and, in case of other streams, by the State in which they are located.

In the third place the capital investment made for stream regulation and the contribution toward flood control by the United States or by an individual State should not ordinarily be made subject to repayment by the benefited parties. Instead of this practice which prevails under the Reclamation Act there should be delivery of the output of water, or power, as the case may be, at wholesale rates to the district or municipality or private concern which can use it to the greatest advantage. Moreover, when there is a choice of the region in which the output is to be used, due weight should be given to the potential development of the remote sparsely populated sections of the country. In the long run the large development of the outlying regions making for a maximum of decentralization will be of greatest benefit to the nation. Furthermore, commitments should not be of such a character as to curtail this development.

If, under the Reclamation Act, each district served with water had been required to pay for the water delivered to it at a fixed wholesale rate and had been left free to distribute the water to the individual farms, all the trouble in recovering the investment and carrying a project during the development period would have been avoided. The cost of carrying a project during this period under such a program, would fall on the Government, as it should, and in that event, this cost could be set down as the Government's contribution to the success of the enterprise.

In the fourth place the owner or original purchaser of lands which are to receive water from a reclamation project, under the Reclamation Act, has received an indirect bonus, which has amounted to 50% or more of the cost of Government works. The giving of a bonus to a farmer or to a group of farmers does not assist farming as an industry. It is unsound in principle and unwise. The prospect of such a bonus immediately adds value to the land, and this value is pocketed by the original owner; it does not benefit his successor. Moreover, while the anticipation of such a bonus by the settler on the lands of the Government project has stimulated farming activities in certain selected localities there has elsewhere been curtailment of agricultural activity. In recent years more than 30 000 000 acres of farm lands have gone out of cultivation. This, of course, must be ascribed to over-production. Careful

consideration should be given, therefore, in the case of every Government-aided land reclamation project, not only to its physical, but also to its economic, aspects, the latter being as much as the former within the province of the investigating engineer.

Thus, for example, the writer who was early connected with the studies which preceded the carrying out of the Sacramento Valley flood-control project in California, has repeatedly pointed out that the inclusion of Sutter Basin in the reclaimed area was a mistake. This basin, which has an area of about 60 000 acres, lies in a pocket between the Sacramento River and the Feather River. Under natural conditions it was subject to inundation by annual high waters of these rivers. However, a large part of the basin land lies so high that the spring flood waters drain off early enough to permit summer crops to be grown. The State should have purchased all the land in this basin and allowed it to be submerged at river flood stages, thus functioning as a retarding basin. The State would then have received some revenue from this area when farmed to summer crops. Instead of following this course, no study of economic advantages and disadvantages was made. The attempted reclamation has burdened the land with a heavy debt and has also increased materially the cost of the control works in the down-stream reaches of the river, because the withdrawal of the basin from the submerged areas has added to the river's peak discharge at all down-stream points.

In the fifth place, land settlement enterprises are local in character and scope, and should never be undertaken by the Federal Government. It should be left to the individual States to organize such enterprises and to make provisions for whatever financial assistance it may be thought wise to extend to the settlers. As already stated, the United States should contribute toward the cost of flood control and, in the case of an output of power or water from the regulation works on an international, interstate, or navigable stream, it should dispose of this output at wholesale, thereby aiding municipalities and districts in securing water supplies, and assuring the success of worthy flood control and other reclamation projects in the several States.

Finally, it is in order to explain why the Engineering Profession must concern itself with such problems as that of shaping the National reclamation policy. The welfare of the country in the years that are to come depends on the utilization and development of its natural resources along well-considered, sane lines. The ordinary citizen is prone to give thought to the immediate situation only. Legislatures are all too frequently driven to conclusions by trading with a hope for local advantage. The engineer with his mind trained to the careful weighing of every feature which has a bearing on his problem, is pre-eminently qualified to ferret out the fundamental principles which should apply in the conservation of water and other natural resources and to determine how best these principles can be applied. It is the duty of the profession to come forward and give the country the benefit of conclusions reached in this and similar matters. The report of the Committee is timely. While the writer believes, as shown in the foregoing comments, that it does not go far enough, nevertheless it is a "long step in the right direction".

S. T. HARDING,* M. AM. SOC. C. E. (by letter).†—This report‡ consists of certain specific conclusions with only limited explanation. The explanatory statement by Mr. Lippincott§ is in many ways a clearer treatment of the subject than the report itself, as it includes the basis for the conclusions. The difficulty of preparing a report by a large committee representing various points of view on a highly controversial subject is obvious.

In the writer's opinion, the most important element in any policy of public aid to irrigation development is the basis on which such use of public funds may be justified and the extent to which aid may be warranted. The usual arguments advanced by those justifying such aid include the benefits of the added taxable resources, increased crop production, and opportunities of insufficiently financed individuals to obtain farm homes. In its first conclusion‖ the Committee apparently recognizes some public contribution as proper. It recommends that such contribution should be in the form of payment of part of the construction costs rather than as an interest waiver. The report, however, is entirely silent on the policy which should govern in determining the extent of the public contribution or its basis of application in specific cases. If it is a proper public policy to spend public funds in land developments where title accrues to the individual land owners, there should be some basis of policy by which the proper extent of such contribution can be determined. The first conclusion as it is now worded can be accepted by both the advocates of full Government aid and by those opposed to any aid. The former can read into the conclusion a large measure of contribution, the latter practically no aid. In the writer's opinion the Committee has omitted the most important point on which definite conclusions might have been influential in relation to National reclamation policies.

That there is need for some definite policy by which the extent of any public contribution to irrigation development can be measured, is shown by the actions of Congress in regard to such aid in recent years. In December, 1924, Congress authorized the revision of contracts with existing projects on the basis of repaying each year on the construction cost, 5% of the average annual gross value per acre of the crops for the preceding ten years. Several contracts were made with existing units on these terms. In March, 1926, such contracts were made with irrigation districts on the Boise Project. The so-called Fact Finders Report¶ gives the average gross crop returns per acre for 1913-22 on the Boise Project, as $38.11, of which 5% would be $1.91, and the construction cost as $80 per acre, of which 11% had been repaid up to 1924. The new contract on these terms extended the repayment period from 20 years to about 40 years. The Kittitas Unit of the Yakima Project was adopted for construction by Congress in 1925, on the same basis of repayment. The published reports of the Bureau of Reclamation show an estimated construction charge of $148.75 per acre. The economic report estimated that

* Prof. of Irrig., Univ. of California; Cons. Civ. Engr., Berkeley, Calif.
† Received by the Secretary, July 1, 1929.
‡ *Proceedings*, Am. Soc. C. E., September, 1928, Papers and Discussions, p. 2097.
§ *Loc. cit.*, May, 1929, Papers and Discussions, p. 1193.
‖ *Loc. cit.*, September, 1928, Papers and Discussions, p. 2098.
¶ Senate Doc. 92, 68th Cong., 1st Session.

96.3 years would be required to repay the construction costs on the "5% of the crops" basis. In 1926, Congress approved the construction of the Owyhee Project at an estimated construction charge for new lands of $160 per acre, with a 40-year interest-free repayment period. Secretary Work reported to the President that in his opinion the project was feasible and the cost would probably be repaid.

The Boise and Owyhee Projects are located in adjacent areas in Western Idaho and Eastern Oregon. They have similar conditions of elevation, climate, crop adaptability, and transportation to markets. The Boise Project is now relatively fully developed. It represents a typical area of its class in regard to crop returns and ability to pay. If the extension of time for the repayment of an $80 per acre charge on the developed Boise Project from 20 to 40 years was justified by the inability of the land owners to pay in a shorter period, there is an apparently obvious inconsistency in expecting the undeveloped Owyhee Project to "pay out" in 40 years with twice as large a rate of payment per acre per year.

In Conclusion (11)* the Committee recommends that interest should be charged on repayment of construction cost and that "the interest rate should be low and the principal payments extended over a long period with no payments on principal during the early years". What is meant by a low interest rate is not stated. A 40-year repayment period without interest is equivalent to paying 2½% interest for 40 years and having the principal written off. Apparently, the Committee had in mind lower interest rates than 2½%, or longer repayment periods than 40 years, if present costs were to be paid in full. If as recommended in Conclusion (1) the contribution should be in principal and not in interest waiver, with interest charged at 4%, one-half the principal would have to be written off in order to amortize the remainder in 40 years without exceeding the rate of payments for the full cost in 40 years without interest.

Projects such as the Owyhee will have construction costs equal to the present value of the lands to be served after they have been prepared for irrigation. If 4% interest is assumed as a proper rate of return to the Government, based on the terms of the Boulder Canyon Project Act, the Government is making a contribution of $80 per acre to bring about irrigation construction on these lands. This apparently represents present National policies. The Committee in its report does not indicate whether it considers present contributions more or less than are justified.

In his explanation of the report, Mr. Mead states† that the measures for aiding settlers in farm development are entirely inadequate and he recommends either that construction of canals for unimproved lands be stopped or additional aid in settlement be given. If a 40-year repayment period on construction costs without interest is not a sufficient attraction to settlers having the necessary funds to develop the lands, in the writer's opinion, Mr. Mead's recommendation for stopping development for unimproved lands is clearly sound. Neither the Committee nor Mr. Mead has presented any basis justi-

* *Proceedings*, Am. Soc. C. E., September, 1928, Papers and Discussions, p. 2099.
† *Loc. cit.*, May, 1929, Papers and Discussions, p. 1208.

fying a larger contribution than that now effective under the present repayment terms. If this contribution is insufficient to attract settlers with means to develop the lands, it may be an indication that settlers having such means have a clearer judgment regarding the economics of land development than those responsible for such projects. Extension of additional public aid in the settlement of irrigated lands has not proved successful where it has been tried by Western States, either in securing the returns from funds loaned by the State or in satisfaction to the settlers.

The preceding comments are not intended as a criticism of the Committee, as the difficulty of reaching a unanimous report on such a subject is fully appreciated. In the writer's opinion more specific conclusions are desirable, even if the attempt to reach them results in divisions within the Committee with subsequent majority and minority reports.

H. H. DARE,* ESQ. (by letter).†—The State of New South Wales covers an area of 310 700 sq. miles. Irrigation projects in this State, as well as elsewhere in Australia, are developed by the State, and the Commonwealth Government is not financially interested, except in one case, the Murray River, 1 300 miles long, at the head of which the Hume Storage Dam (2 000 000 acre-ft. capacity) is being constructed at the joint expense of the Commonwealth and of each of the three States of New South Wales, Victoria, and South Australia, which are watered by the river.

Under the New South Wales Act, which was passed before 1909, the right to the control and use of all water in all rivers and lakes in that State which flow through, past, or are situated within, the land of two or more occupiers, is vested in the Crown. No person is permitted to obstruct the flow, or change the course, of any such stream, or to divert water therefrom, unless a license has first been obtained. Before such a license is granted a public inquiry is held, at which it is possible for any interested person to appear in support of, or opposition to, the granting of the license. It should be noted that the State does not claim ownership in water, but merely controls the streams in the interests of the riparian holders, the license fees collected being sufficient only to cover the cost of administration.

Apart from irrigation of their own lands by landholders, irrigation development in New South Wales is of three classes, namely,

(1) Irrigation Trust Projects, in which the works are constructed by the State, and the cost is repaid by the landholders with interest over a period of years.
(2) Private Irrigation Projects, in which works for the supply of two or more landholders are constructed and paid for by the owner of land fronting a river or lake; and
(3) State Irrigation Projects.

In the case of Classes (1) and (2) the management is local, and the Water Conservation and Irrigation Commission exercises a general supervision only, and makes no advances to settlers. With regard to Class (3), the Commission may initiate and carry on a State Irrigation Project, subject to the provision

* Commr., Water Conservation and Irrig. Comm., New South Wales, Sydney, New South Wales, Australia.

† Received by the Secretary, July 29, 1929.

of the necessary funds by Parliament. The process involves four steps as follows:

(*a*) Acquire all the land within the boundaries of the scheme.
(*b*) Construct and maintain the necessary roads, etc., as well as works for the storage and distribution of water.
(*c*) Sub-divide the land into irrigation farms, allot water rights to each farm, and determine the price of such water rights.
(*d*) Make available the blocks for application by means of:
(1) Purchase in fee simple (freehold), with payments including principal and interest at 6½%, extending over 36½ years; or,
(2) Lease (in recent schemes as perpetual leases), with an annual rental as gazetted, for the first twenty-five years and, subsequently, as may be determined to be the fair market annual rental value, irrespective of any improvements.

Concerning Step (*d*), water is either measured at the outlet to each block on a per acre-foot basis; or, in some cases, where meters are not installed, it is charged as a fixed rate per acre per year, covering a specified number of waterings, with an extra charge per acre for special waterings, if required.

Conditions in New South Wales are different from those in the United States, inasmuch as the control of water is vested in the State, and the Commonwealth Government has no authority or interest in irrigation development, except as a fourth partner in the construction of the head storage and locks and weirs on the Murray River. The irrigated lands developed under State enterprise are already owned by the State, or are acquired by it before subdivision.

With regard to the payment of interest on the cost of construction in the case of Trust Projects (Class (1)), which is relatively small, the annual repayments to the State include the payment of interest and those made to a sinking fund to repay the cost of construction over a period of years.

In the case of the Murrumbidgee Scheme, which is the largest of the State Irrigation Projects, the principle was laid down that the cost of the works should not be repaid, but that the interest thereon, together with certain other charges, should be recoverable in the form of rent and water rates. At that time the tenure was entirely by perpetual leasehold, but, with the subsequent change of Government policy, under which conversion to freehold is permitted, the money received from such conversion is allotted partly as interest and partly as principal toward reduction of the capital debt. The values on re-appraisal of the land held under perpetual lease after the expiration of the first period of 25 years and subsequent periods of 20 years, and on conversion to freehold, are fixed by the Land and Valuation Court, if the settlers should make application therefor. Since these values are dependent to a great extent on the marketing position, they may not represent the figure originally estimated for the cost of land plus works. Such difference in values will represent State aid to irrigated agriculture.

In connection with the Murray River, a different principle will probably be followed, inasmuch as there has been no proposal to recover from irrigation settlers in this State the proportion of the cost of construction of the head

storage, which is treated as a National work for which the State must pay. The cost of distributary works, however, will be recoverable from the settlers.

It has been argued that free head-works should be a general charge on the community, as in the case of bar harbors and certain other public works, the benefit to the State being realized in terms of new settlers gained and in increased general prosperity. There is no doubt that on the rivers in New South Wales the regulated flow resulting from a head storage is of benefit to all the riparian holders, most of whom do not hold an irrigation block included in any State Project. To apply the principle of "betterment" to these holders would be difficult, and it is not possible under existing law. This matter is now receiving consideration.

The experience in New South Wales in regard to assisting settlers on irrigation farms is in accord with that in the United States, as referred to in Mr. Mead's paper,* namely, that:

(1) Due to the increased cost of developing farms, and to the uncertain position as to markets for their produce, there are only a few settlers offering to enter irrigated lands; and
(2) Such settlers as are applying for farms have in very few cases sufficient capital to carry on without assistance. Such assistance is rendered by the State, either through the Water Conservation and Irrigation Commission, or through the rural bank, which raises its money by the same methods as an ordinary trading bank. This money has to be used for rural development purposes, either by way of advances to individual settlers or to co-operative groups.

The bank makes advances on irrigation farms by overdraft on current account, repayable on demand, with interest (at present) at 6½ per cent. Settlers must have the means to effect some improvements before the bank will assist. Advances are limited to certain percentages of the value, depending on the nature of the improvements. Irrigation settlers are not offered more advantageous conditions than those on non-irrigated holdings. Although it is now the policy to have the bank arrange for the financing of irrigation settlers, there are still a large number assisted by the Commission. These include some of the earlier settlers, and also the soldier settlers who took up farms without any capital, in accordance with the land settlement policy adopted at the termination of the World War. Following on the increase in the cost of production, and the fall in market values with regard to some of the products, a re-adjustment is being made in the case of the soldier and, also, of some of the civilian irrigation settlers.

The Coomealla Area, which is the largest irrigation project of the Water Conservation and Irrigation Commission, is situated on the Murray River. It embraces about 110 holdings, each ranging from about 15 to 25 acres, and they are used exclusively for fruit growing, principally vine fruit and citrus. Assistance is given by the Commission by only charging the full water rate at the expiration of five years after the granting of each block. In the intervening years a reduced water rate, progressing from one-fifth of the total upward, is charged. On application by the settler the payments of all moneys

* *Proceedings*, Am. Soc. C. E., May, 1929, Papers and Discussions, p. 1208.

due for rent or instalment of purchase money and other charges, is suspended for periods extending to 3 or 4 years from the date of granting the block, according to the class of planting, and subject to a charge for interest at 6½ per cent. At the end of this period the amounts thus suspended are compounded, and the settler is then required to enter into negotiations with the rural bank, or to obtain necessary financial assistance otherwise, to pay off the money due the Commission. During the period up to which the debt is taken over by the bank, the Commission makes advances against improvements on the same scale as that allowed by the rural bank, and before any such advances are made a valuation is prepared of the settler's assets, so that at all times the State is in the favorable position of having a secured asset for any money advanced. These advances are also suspended, with interest at 6½%, on the same understanding with regard to their being taken over by the bank at the date fixed.

Blocks were granted to settlers on this area in 1926, and most of the blocks have been allotted in about three years. The settlers have been either the sons of landholders from the surrounding district, or laborers previously employed in irrigation work. In most instances these men have had very limited capital, but they have had experience, and the result has been very satisfactory, because most of the improvements have been carried out expeditiously, and at low cost. The average of the advances made to date (1929) only represent a little more than £100 ($485) per settler. During the 1928 season the production of vine fruit from a number of the blocks has been quite considerable. The bank will be called upon in 1929 to take over the debts of some of the earlier settlers. It is willing to do so because the assets are substantial.

If the marketing position remains satisfactory, the Coomealla Settlement gives promise of being a successful one, but without State aid the position of the majority of the settlers would have been impossible. It has been only by judicious assistance that these men have been able to make good, and, were it not for State assistance, very few of the blocks on the settlement would have been developed. Such assistance, it should be noted, has been made only after a careful investigation of the assets on each block, and after a report from the local manager, who is in close touch with the settlers, and is in a position to form an estimate of the personal qualifications of each. With such safeguards the writer is of the opinion that the principle of State aid is a sound one.

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EXPERIMENTS ON DISCHARGE OVER SPILLWAYS AND MODELS, KEOKUK DAM

Discussion*

By Dr. Ing. Ludwig A. Ott.

Dr. Ing. Ludwig A. Ott† (by letter).‡—This paper is of interest to the writer because it is the first that describes a case in which discharge measurements with current meters have been made under such extraordinary conditions and with such satisfactory results.

The particular difficulties in the measurements at the Keokuk Dam were caused by the presence of very high water velocities combined with great depths and oblique currents.

In high water velocities and at great depths the mechanical stress on a current-meter rod of the usual circular cross-section is far greater than one is inclined to believe, if one has not made this problem a special study. Besides, even in uniformly smooth currents high lateral stress may frequently occur, that may cause intense vibrations of the rod.§ Rods of stream-line shaped cross-section are by far the most serviceable. Under equal conditions of water velocity and width of section, the pressure on such a rod is only about one-fifth that on a rod of circular section.

In regard to the current meters themselves, the Ott meters, used in the researches, will withstand the greatest velocities without danger of injury. The most noteworthy examples in this respect are perhaps the measurements reported by F. Kuntschen.‖ The meter used in these experiments with full success at a water velocity of 42 ft. per sec. (corresponding to more than 6 000

* Discussion of the paper by Floyd A. Nagler and Albion Davis, Members, Am. Soc. C. E., continued from August, 1929, *Proceedings*.

† Math. Mech. Institut, Kempten, Bavaria, Germany.

‡ Received by the Secretary, May 29, 1929.

§ See F. Eisner, "Widerstandsmessungen an umströmten Zylindern", *Mitteilungen* der preuss. Versuchsanstalt für Wasserbau und Schiffbau, No. 4, Berlin, 1929.

‖ "Le moulinet hydrométrique et la mésure de courants très rapides," Communications, Suisse Service Fédéral des Eaux, No. 18, Berne, 1926.

rev. per min.) was of the same type as the instrument used at the University of Iowa for velocity measurements in its experiments on the weir model.*

In the case of extended measurements under very high stress, however, the particular Ott meters used at the Keokuk Dam,† are scarcely the most serviceable, because their axle bearings are not devised for such an extraordinary stress for long hours. It would be better to use an Ott meter of a type provided with steel ball-bearings running in an oil bath. This construction permits using the meter with confidence in extremely high velocities without affecting in the least its sensitiveness for a current as slow as about 0.15 ft. per sec.

A third difficulty in making measurements at the Keokuk Dam (which was also overcome satisfactorily) was the existing oblique currents.

The execution of measurements in oblique currents is the more simple and exact, the more closely the behavior of the screw follows the cosine law. Experience has taught that in this respect conical screws with two blades give less accurate results than cylindrical screws with three blades attached to spokes, as has also been proved by the U. S. Bureau of Standards ratings of the meters used. The narrowness of space alone was responsible for the use of the 2-blade, instead of the 3-blade screw, 7 in. in diameter. In connection with the meter equipped with ball-bearings in an oil bath, cylindrical screws, 4 in. in diameter, are furnished which show a particularly small deviation from the cosine law. At an angle of 20° between the direction of flow and the axle of the meter, the correction to be added to the determined velocity (see Fig. 6‡) is only 2 per cent.

In making the statement that the measurements at the Keokuk Dam might have been carried out more advantageously by the use of a rod with a streamline shaped section and an Ott meter of different type with a 3-blade cylindrical screw, 4 in. in diameter, the writer of course, intends no reflection on the accuracy of the results obtained by the means used. On the contrary, he considers them to be highly satisfactory.

The comparative measurements at the Keokuk Dam and on a model of the spillways are similar to measurements carried out by the Research Institute for Hydraulic and Hydro-Electric Structures (Forschungsinstitut für Wasserbau und Wasserkraft), in Munich, Germany, at the spillways of the Mittlere Isar River and on a model one-fiftieth of the full size.§ In this case the comparison of the results obtained on models and at the actual structure also produced a very satisfactory confirmation of the model rule.

At the writer's request, Dr. Ing. O. Kirschmer, Chief Engineer of the Research Institute for Hydraulic and Hydro-Electric Structures, has compared the results of his measurements with those at the Keokuk Dam with satisfactory results, as follows:

During its experiments the Research Institute for Hydraulic and Hydro-Electric Structures has affected many changes in the forms of the weirs.

* *Proceedings*, Am. Soc. C. E., February, 1929, Papers and Discussions, p. 341.

† *Loc. cit.*, p. 312.

‡ *Loc. cit.*, p. 317.

§ A report on these experiments is contained in "Hydraulic Laboratory Practice" (pub. by the Am. Soc. Mech. Engrs.), pp. 442-453, under the heading "Experiments to Determine the Discharge Coefficients of Certain Weirs with Round Crests".

There was little difficulty in comparing the results obtained with those at the Keokuk Dam. After reducing the weir coefficient determined by the Research Institute to existing conditions at the dam, the results obtained in both studies show a very satisfactory agreement. All the results determined on the model at the University of Iowa lay well within the range of "scattering" values observed in the experiments of the Research Institute (see shaded area in Fig. 27).

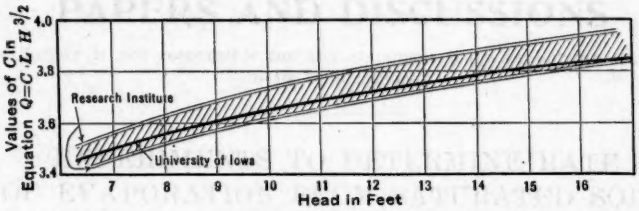

FIG. 28.—COMPARISON OF THE MODEL EXPERIMENTS OF THE UNIVERSITY OF IOWA AND THE RESEARCH INSTITUTE FOR HYDRAULIC AND WATER POWER ENGINEERING, MUNICH, GERMANY.

After very extended studies the Institute has reached the conclusion that on weirs certain variations in the flow condition will arise (instability of the current), and that, owing to this circumstance, various coefficients may be determined under fixed conditions at one and the same weir. For this reason, Dr. Kirschmer has not drawn in Fig. 27 a reduced or mean curve of weir coefficients, but has shown only the upper and lower limit between which the coefficients vary, owing to the unstable condition of the current. If one draws a mean curve within these limits, there remains a negligible difference between this curve and the one obtained by the authors which, perhaps, may be accounted for by the different methods of measuring (volumetric measurements at the Institute and Bazin weir measurements at the University of Iowa). Incidentally, in Germany the Bazin weir formula is now scarcely used for the computation of discharges.

The experiments at the Keokuk Dam can be compared further with those of the Research Institute at the spillway of the Mittlere Isar River with regard to the influence of obstructions, such as piers. In this case there is likewise an agreement between the authors' and Dr. Kirschmer's experiments. This influence on the weir coefficients appears to be somewhat less in the experiments of the Institute than at the Keokuk Dam, but it must be remembered that at the Mittlere Isar the thickness of the piers is also less in proportion to the width of the spillways at the Keokuk Dam.

The agreement between the model experiments and the measurement at the actual structure obtained on both parts may also be considered an indirect proof of the reliability of the current-meter method of measuring discharge. By virtue of its previous experience with regard to current-meter measurements the Research Institute is convinced that, with meters of suitable construction, great accuracy of results can be obtained even in very turbulent currents. In order to gain conclusive evidence as to the absolute accuracy of current-meter measurements, the Institute intends to make extended experiments by comparison with the volumetric method. For this purpose the

Institute has provided for the erection at its laboratory near the Walchensee a measuring tank with a capacity of 53 000 cu. ft., which will soon be put to use.*

Investigations concerning the accuracy of the various methods of discharge measurements form an essential item in the program of the Research Institute.

* For a description of the Research Institute of Hydraulic and Hydro-Electric Structures, see "The Research Institute for Hydraulic and Hydro-Electric Structures", by B. R. Van Lear. *Mechanical Engineering*, Vol. 50, No. 8, August, 1928, pp. 607-611.

AMERICAN SOCIETY OF CIVIL ENGINEERS
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PAPERS AND DISCUSSIONS

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EXPERIMENTS TO DETERMINE RATE OF EVAPORATION FROM SATURATED SOILS AND RIVER-BED SANDS

Discussion*

BY MESSRS. SAMUEL FORTIER, A. L. SONDEREGGER, AND HARRY F. BLANEY.

SAMUEL FORTIER,† M. AM. SOC. C. E. (by letter).‡—The United States is mainly indebted to two Federal agencies for its knowledge of water supplies. The U. S. Weather Bureau measures and records the precipitation as it falls from the clouds in the form of rain and snow, and the U. S. Geological Survey measures and records the quantity of water that flows to the sea. The incoming waters derived from precipitation, particularly throughout the western half of the United States, greatly exceed the outgoing waters conveyed in stream channels. There are large areas in the West in which no water derived from precipitation ever reaches the ocean in the form of run-off. This is true of the Great Basin comprising an area, exclusive of the Imperial Valley, of 138 000 000 acres. A near approach to this condition is also found in the basin of the Colorado River containing 155 000 000 acres, in which the normal run-off is less than 0.8 in. over the surface drained.

These references and others which might be given, show that in that vast region comprised in the seventeen Western States, only a small part of the total precipitation is carried off by streams, the greater part being converted into vapor which is soon wafted to distant localities by the prevailing winds. No Federal agency has been specifically charged with the collection of data having for its objective the determination of the quantities of water transpired from native vegetation and evaporated from uncultivated soil surfaces,

* This discussion (of the paper by Ralph L. Parshall, Assoc. M. Am. Soc. C. E., published in April, 1929, *Proceedings*, but not presented at any meeting of the Society), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

† Prin. Irrig. Engr., Div. of Agricultural Eng., U. S. Dept. of Agriculture, Berkeley, Calif.

‡ Received by the Secretary, May 27, 1929.

for the apparent reason that until recently those who live in the humid part of the country have not been generally interested in the subject. On the other hand, to those whose occupation and abodes are in the arid and semi-arid regions—and they comprise one-fifth of the total—the subject is of vital concern. While they do not claim to be able to control, to any great extent, transpiration from plants or evaporation from soils, yet they do require a general knowledge of this enormous loss of water in order to plan aright for the utilization to the fullest extent possible of what remains of the water supplies.

Until recently the main water problems of the Nation have been confined to the territory west of the Missouri River because water was scarce, and because much more was needed for agricultural purposes. Westerners discuss water, investigate it, litigate over it, and end by criticizing the water decisions of the Courts. Now, however, many Western water controversies are having their counterparts in the Eastern States. The rapid increase in the use of water for hydro-electric plants and municipalities has enhanced its value and has led corporations and municipal and State Governments to transfer supplies from one drainage basin to another and from one State to another. This is quite enough to start trouble, and no one is wise enough to predict when or how it will end. In view of this situation, which is certain to become worse, and in the interests of public welfare generally, increased Federal and State funds might profitably be expended in acquiring more complete and reliable data concerning water supplies. Western experience has demonstrated that it is much cheaper and preferable in many ways to employ engineers to investigate water problems than lawyers to litigate over them.

A field that is practically unexplored and one that should be investigated at an early date is to be found in the wide gap between the accumulated knowledge available concerning precipitation on the one hand and run-off on the other. To call attention to but two vacant spaces in this unexplored field of knowledge, there still remain in this country about 470 000 000 acres of virgin forests and cut-over and burned-over forested areas. Does this vast area transpire annually, on an average, 1 acre-ft., 2 acre-ft., or some other quantity of water per acre? Nobody knows and nobody has ever tried to find out. Furthermore, there are about 658 000 000 acres of unimproved pasture and range land. What is the loss of water by transpiration and evaporation from this area? Nobody knows.

In many respects, transpiration of water from plants differs from evaporation of water from soils, each having distinct and separate functions to perform, but in studying water supplies and their utilization it is seldom practical to consider them apart. Four drops of water are lodged in soil side by side. Three drops are absorbed by the rootlet of a plant, drawn up through its stem, and transpired by its foliage. The other drop is evaporated directly from the soil. All four reach the clouds, but by different routes. If one part of the land surface of the United States were bare soil and the other covered with vegetation, the rate and extent of evaporation from the bare part might be determined, but the almost inseparable twin problems of transpiration and evaporation would be found in the other. As a matter of fact there is very

little soil in which plants of one kind or another do not grow at least a part of each year, and the quantity of water which is transpired by plants from a unit of such surfaces cannot well be segregated from that evaporated from bare soil in their immediate vicinity.

Since 1898 the Congress of the United States has appropriated relatively small sums of money to the Division of Agricultural Engineering for irrigation investigations. By co-operation with Western agricultural experiment stations and other State agencies, it has been possible, through the increased funds thereby made available, to carry on such studies in all the Western States. Several thousand experiments have been conducted in widely separated localities to determine the water requirements of all the most common field crops and the quantities of supplemental irrigation water needed for their proper growth. As a result of these investigations it is now possible to define with reasonable accuracy the irrigation requirement of a farm in any one of the Western States, provided knowledge is available of its location, climate, extent of irrigable area, character of soil, and the crops to be grown. Since, however, the irrigated land in the seventeen Western States constitutes less than 2% of the total, a knowledge of the quantity of water transpired and evaporated from irrigated farms does not help much in the solution of what takes place in respect to these natural functions on the remaining 98%, particularly as little or no data are available on which to base an estimate. This is true of native grass lands and lands covered with brush, shrubs, and virgin forests. It is likewise true to nearly the same extent of arable lands which are dry farmed. An erroneous impression prevails in attributing to soil evaporation the greater part of the vaporized waters escaping from uncultivated lands on which native plants, including forests, grow, whereas actually it is the native vegetation that transpires by far the greater part.

In passing from a consideration of the water yield of a Western drainage basin to the water problems of the irrigator, there are many ways in which evaporation from soils affects the irrigator's profits. In growing cultivated crops, such as orchard trees, cotton, sugar beets, or potatoes, the greater part of the ground surface is not only bare but is periodically watered and a large percentage of the water applied escapes as vapor into the atmosphere. To cite a single case, the average annual cost of irrigation water applied to the lands of 111 mutual water companies in California is $15.50 per acre. The yearly cost of applying this water is about $2.50 per acre, or $18.00 per acre in all. If 30% of this water is evaporated from bare soil, it represents a considerable loss to the grower. In an effort to find ways and means to lessen this and similar losses, the writer, aided by his associates and collaborators, sought to determine some of the underlying principles of soil evaporation. Dr. John A. Widtsoe, when President of the Agricultural College of Utah, and W. W. McLaughlin, now Associate Chief of the Federal Division of Agricultural Engineering, were the first to ascertain quantitatively by controlled experiments in tanks that the more moisture the soil contained the greater was the evaporation.

A series of experiments conducted by the writer confirmed this fact and led to the use of deeper furrows in applying water to cultivated crops. In the

case of the orchard groves of California, the top layer of soil from 6 to 9 in. in depth, although it is the most fertile and best aerated part of the soil column of the root zone, serves mainly as a blanket to protect the moist soil beneath. Orchard roots which are lured into this zone by winter rains perish from drought during the long rainless summer. The writer also determined by experiments that the factors of temperature, wind movement, and humidity which influence the rate of evaporation from water surfaces were applicable to soil surfaces. It is well, however, to keep in mind that the governing factor in evaporation from soils consists in the percentage of moisture in the top layer. Under like conditions saturated soils may be said to evaporate about 90% of that from a water surface, but with the soil moisture at the wilting point of plants the evaporation may be reduced to little or nothing.

Another phase of evaporation which concerns rural communities of the Western States, is to be found in the condition of Western streams and their flood-plains during low-water periods. Some Western stream channels carry water so seldom that their natural function is not always apparent to the casual observer. Other streams, like the Platte, Rio Grande, and Arkansas, in the late summer months carry a small volume of water spread out in a thin sheet, while the greater part of the flood-plain is either dry or partly moistened. It occurred to the writer that if the loss of evaporated water from the shallow water surface as well as from the bare river bed material could be determined with a fair degree of accuracy, a basis could be laid to determine the economic feasibility of conveying the flow of the late summer months, either in an artificial canal or within a relatively small part of the flood-plain channel.

With this objective in mind, advantage was taken of a rather far-flung organization to collect twenty-seven typical 40-lb. samples of river-bed materials from each of sixteen stream beds. A mechanical analysis was made of each sample, and the percentage of pore space was also determined. The equipment was inadequate to make a separate evaporation test of each sample and, for this reason, the samples were combined into four groups ranging from the coarsest to the finest material. This grouped material was placed in double water-jacketed tanks of which the inner, with a few exceptions, was 23.5 in. in diameter and 48 in. deep. In an effort to duplicate Western stream-bed conditions in low-water periods, the water-table in each series of tank experiments was maintained at distances from the surface of 3 in., 12 in., and 24 in., and occasionally at other distances. Where the water-table was 3 in. below the surface, the results were fairly uniform for the different materials, varying from 66 to 77% compared with 100% from a water surface; but at depths of 12 and 24 in., the variations were greater. The experiments were carried on at the Denver, Colo., Field Laboratory and were in direct charge of the writer's assistant, the late R. B. Sleight, Assoc. M. Am. Soc. C. E.*

In this brief discussion of Mr. Parshall's paper, the writer has endeavored to show the relationship between evaporation from soils and transpiration from plants, and to give a few reasons for the acquisition of more knowledge of both branches of this twin problem. D. C. Henny, M. Am. Soc. C. E., and his associate members of the Special Committee on Irrigation Hydraulics,

* *Journal of Agricultural Research*, Vol. 10, No. 5, 1917.

deserve commendation for their efforts to collect the scattered records pertaining to evaporation in order that around this nucleus may accumulate a larger body of reliable information. In the writer's opinion no one is better qualified by training and experience than the author to advance the study of the subject through another stage of progress.

A. L. SONDEREGGER,* M. AM. SOC. C. E. (by letter).†—This paper describes a method of determining evaporation which apparently is vastly superior to the cumbersome method of finding losses by weighing the soil mass in tanks, and also to the method of maintaining a fixed water-table by daily closing. The experiments themselves present a valuable addition to the study of evaporation losses from moist areas.

In many regions of the semi-arid West, the elimination of evaporation from moist areas will form the last link in the chain of efforts to salvage the losses that pertain to the natural disposal of water supply. The factors which affect the problem are the total volume of evaporation losses available for recovery and the depth to which the water-tables must be lowered, with different soils, in order to stop capillary action.

As far as the experiments described by the author are concerned, it is apparent that the adjustment of the Mariotte flask to the special conditions presented by each test tank offers the most difficult problem with which the experimenter has to contend.

The results graphically shown on Fig. 4‡ permit of practical application. The sharp descent of the curve for the two extremes in soils—adobe and coarse sand—would indicate that, with a water-table 2 ft. below the surface, evaporation losses are eliminated. In clay soils, capillary action is effective, but very slow, while in the coarse sand, capillary tubes are short and interrupted. On the other hand, the high rate maintained by Rocky Ford sand, which occupies a position about half way between the two extremes, leads to the conclusion that the mixture of materials was such as to produce what might be called "optimum" capillary action, both as to time and rate. It must be remembered, however, that these soils were disturbed, having been removed from their original location.

In order to complete the evaporation curve, deeper tanks are required. Experiments to determine evapo-transpiration losses from undisturbed cores are now being made by A. A. Young, Assoc. M. Am. Soc. C. E., under the supervision of H. F. Blaney, Assoc. M. Am. Soc. C. E., of the Division of Agricultural Engineering of the U. S. Department of Agriculture, in connection with the Santa Ana River investigation in Southern California, at San Bernardino, with tanks 23 in. in diameter and 40 in. in depth; and at Santa Ana with tanks 23 in. in diameter and 72 in. in depth, all of which are operated by the use of the Mariotte principle.

The effect of applying the water by sprinkling the soil surface has been to increase the evaporation losses two to three times as compared with a stationary water plane fed from below. While these results may not be directly

* Cons. Engr., Los Angeles, Calif.
† Received by the Secretary, June 22, 1929.
‡ *Proceedings*, Am. Soc. C. E., April, 1929, Papers and Discussions, p. 854.

applicable to a comparison between irrigation by sprinkling and by furrows, nevertheless they indicate that, of necessity, the evaporation losses must be much larger for overhead application.

Overhead sprinkling systems for citrus and avocado orchards are installed in Southern California on increasing areas and at costs of as much as $300 per acre, yet sometimes apparently with little regard for the increased evaporation losses. In some cases this form of irrigation has not been successful. Some of the failures must be attributed to the fact that, despite comparatively heavy soils, no additional water supply was provided. With an abundant supply, overhead irrigation can be carried on successfully, provided soils are not too heavy. In such localities the trees appear to be healthy and are bearing abundantly. For steep slopes, this system presents a practical method of overcoming the washing of soils and excessive waste. Tests to establish evaporation losses from sprinkling systems, for different soils, in comparison with furrow irrigation, would be of general interest.

The author has given the mechanical analysis of soils in terms of the percentage of the complete sample. This method may be the most accurate, and, for very coarse materials, the only one, but it does not lend itself to a quick interpretation, except possibly by an experimenter.

Soil designation by the moisture equivalent is becoming a matter of common practice and permits of a ready comparison of different samples. In experiments of the character described, the results of which lend themselves more or less directly to the solution of concrete problems, the writer would suggest that the moisture equivalent and, with undisturbed cores, also the volume weight and porosity of the samples, be given.

The author refers to experiment* of Professor F. H. King by which it was proved that for dry surface soils the effect of an application of water, which does not produce percolation, is to strengthen capillary flow so that,

"The surface foot draws upon the deeper soil moisture at a more rapid rate than before, causing a translocation of the lower soil moisture, the deeper soil becoming measurably dryer soon after such a rain than it was before it, while the surface foot is found to contain more water than has fallen on it."

This phenomenon is believed to be responsible for the drying out of bare soils to great depths during periods when rain storms are generally light and even when the maximum storm of the season does not exceed 3 in. in amount.

HARRY F. BLANEY,† ASSOC. M. AM. SOC. C. E. (by letter).‡—In Southern California evaporation losses from soils and river-bed sands are an important factor in water supply studies and could be estimated fairly accurately, provided basic data, such as the author describes, were available. Data of this nature may also prove valuable in estimating surface evaporation from soils immediately after an irrigation. The Division of Agricultural Engineering, Bureau of Public Roads, U. S. Department of Agriculture, in co-operation with the California Department of Public Works, is now making a study of evaporation losses from moist lands in the Santa Ana River area of Southern

* "The Soil," by F. H. King, 1898, p. 196.

† Irrig. Engr., Div. of Agricultural Eng., U. S. Dept. of Agriculture, Los Angeles, Calif.

‡ Received by the Secretary, July 15, 1929.

California. In this work Mr. Parshall's method of using the Mariotte principle has been extended to tanks 6 ft. in depth, with a water-table maintained at 2 ft. and 4 ft. below the ground surface. The tanks are filled with undisturbed soil and are operated in triplicate.

During 1919 the writer continued the evaporation studies* begun by the late R. B. Sleight, Assoc. M. Am. Soc. C. E., at Denver, Colo., under the direction of Samuel Fortier, M. Am. Soc. C. E., and was very much impressed with the effect of the size of tanks on the depth of evaporation from a water surface. In extending evaporation data obtained from small soil tanks to larger land areas the engineer is confronted with the problem of selecting a proper reduction factor. Very few data are available for accomplishing this. However, factors have been established for extending evaporation data from various sizes of tanks to larger water surfaces. Mr. Sleight† concluded that evaporation data from circular tanks at least 2 ft. in depth may be quite safely extended to large open water surfaces under exactly the same conditions of wind, air temperatures, and relative humidity, by multiplying the evaporation depth from a 2-ft. tank by 0.77, a 4-ft. tank by 0.84, a 6-ft. tank by 0.90, a 9-ft. tank by 0.98 and a 12-ft. tank by 0.99." The writer believes that a somewhat similar relation exists between evaporation from various sizes of tanks containing saturated soils, and that if no better data are available the factors mentioned may be used in estimating the evaporation losses from larger land areas where the water-table is near enough to the surface to saturate the soil.

The data which Mr. Parshall presents in Table 4,‡ for Tank 17, August 21 to September 25, is of value in showing the surface evaporation loss after rain storms. Tank 17 generally lost a very small amount before the sprinkling started. During this period no water was supplied from the Mariotte tank. About 0.3 to 0.5 in. was applied per week and most of this was evaporated. It is probable that the water applied by sprinkling would not immediately drain down to the water-table, but that at least five days would elapse before equilibrium was again established and, during this time, surface evaporation of the sprinkled water would be taking place. If the quantities applied were small enough, none would move down to the water-table.

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* *Transactions*, Am. Soc. C. E., Vol. 90 (1927), p. 303.

† "Evaporation from the Surfaces of Water and River-Bed Materials," *Journal of Agricultural Research*, Vol. X, 1917, No. 5, p. 237.

‡ *Proceedings*, Am. Soc. C. E., April, 1929, Papers and Discussions, p. 852.

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The data which Mr. Parshall presents in Table 4.2 for Tank 17, August 21 to September 20, is of value in showing the surface evaporation loss after rain storms. Tank 17 generally lost a very small amount before the sprinkling started. During this period no water was supplied from the Mariotte flask. About 0.3 to 0.5 in. was applied per week and most of this was evaporated. It is probable that the water applied by sprinkling would not immediately drain down to the water-table, but that at least five days would elapse before capillarity was again established and, during this time, surface evaporation of the sprinkled water would be taking place. If the quantities applied were small enough, none would move down to the water-table.

[1] "Evaporation from the Surfaces of Water and River-Bed Materials," Journal of Agricultural Research, Vol. XI, 1917, No. 5, p. 227.

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PROBLEMS CONCERNING ELASTIC STABILITY IN STRUCTURES

Discussion*

By Messrs. H. M. Westergaard and William Hovgaard.

H. M. Westergaard,† M. Am. Soc. C. E.—The author has written a number of scientific papers on the subject of elastic stability, and has given a notable development to the technique of solving problems in this field. He has used extensively a general principle of equilibrium which is based on the variations of the potential energy of the deformed structure and the loads. The systematic use of this principle has been fruitful.‡ The particular problems treated in this paper are unquestionably important.

The principle of variation of the potential energy lends itself to mathematically exact solutions of the simpler problems and to approximate solutions of the more complex ones. Generally speaking, when approximations are admitted, the problems shift from mathematics to physics. In this connection a remark by the late C. Christiansen, Professor of Physics in Copenhagen, deserves to be quoted: "There is the difference", he said, "between mathematics and physics, that in mathematics everything is right and in physics everything is wrong." A method which belongs partly in mathematics and partly where "everything is wrong" requires mathematical technique of course, but it also requires sound judgment, as well as common sense on the part of the engineer or the physicist. These requirements are not easy to meet, but in the speaker's opinion, the paper meets both of them. It may be added that a sound judgment is expected also of those who are to interpret the results for the purpose of structural design.

* This discussion (of the paper by S. Timoshenko, Esq., presented at the meeting of the Structural Division, Columbus, Ohio, October 13, 1927, and published in April, 1929, *Proceedings*), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

† Prof. of Theoretical and Applied Mechanics, Univ. of Illinois, Urbana, Ill.

‡ For extensive bibliographical information, see the chapter by Professor Timoshenko under the heading, "Stabilitätsprobleme der Elastizität," in Handbuch der physikalischen und technischen Mechanik, v. 4, Lieferung 1, 1929, pp. 81–145.

The paper contains definite answers to definite questions which may be asked by the structural engineer. Answers are stated in terms of tabulated numerical values which are to be substituted in simple algebraic expressions.

After receiving these definite answers, it is reasonable to ask where one stands. This issue may be discussed by referring to the most familiar problem of elastic stability. Fig. 14 (*a*) shows diagrams of load and deflections for a slender column. The combination, *O A B*, represents what would happen if the column were perfectly straight, homogeneous, and centrally loaded. In a test, in which these conditions of the ideal column cannot be entirely satisfied, one would find, perhaps, the curve, *O C D*. These diagrams are characteristic of the buckling of slender structures in general. The ordinate, *O A*, represents the computed critical load. Under practical conditions one must expect a curve like *O C D*. It may be stated, however, that knowledge of the position of the horizontal line, *A B*, helps in forming a judgment as to the position of the curve, *C D*.

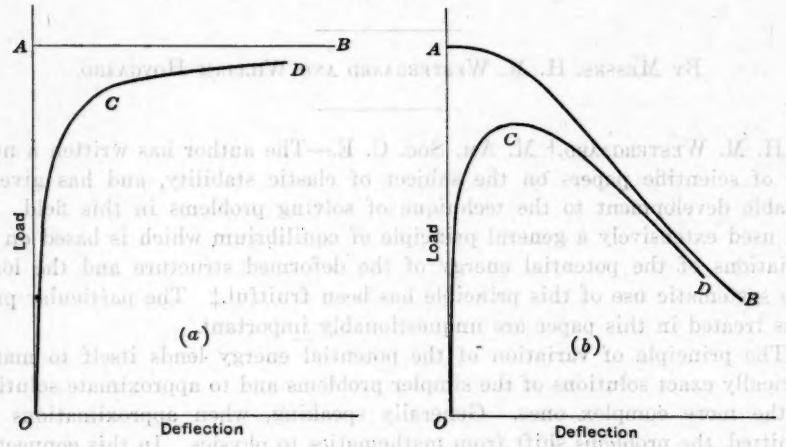

FIG. 14.

If the column is not particularly slender, but of medium length, one may obtain the curves shown in Fig. 14 (*b*): *O A B* for the ideal column, the part, *A B*, being curved because some of the stresses exceed the proportional limit; and *O C D* in a test, or in practice. Grant that the critical load, *O A*, has been computed, in spite of the difficulties which accompany a reduced modulus of elasticity. It is the ordinate of the apex, *C*, which is of practical interest. This true maximum load may be considerably less than the load, *O A*, obtained under idealized conditions. Th. von Kármán studied these relations for steel columns in an investigation published in 1910.*

The diagrams in Fig. 14 (*b*) are characteristic of the buckling of the less slender structures in general. Professor Timoshenko refers occasionally to

* Th. von Kármán, "Untersuchungen über Knickfestigkeit," *Forschungsarbeiten* auf dem Gebiete des Ingenieurwesens, Heft 81, 1910; see, also, M. Ros, *Proceedings*, Second International Cong. for Applied Mechanics, Zürich, 1926 (pub. 1927), p. 368; and a paper by William R. Osgood, Assoc. M. Am. Soc. C. E., and the speaker, *Transactions*, Am. Soc. Mech. Engrs., v. 50, 1928, No. 17, p. 65.

these less slender structures. It is true that an estimate of the position of Point A in Fig. 14(b) may help in forming a judgment as to the position of Point C. The problem involved here still represents a field for future research, both experimental and analytical.

The problems of elastic stability of slender structures, in which stresses exceeding the proportional limit need not be considered, are, of course, important for their own sake. It is a peculiar circumstance, however, that a study of the critical loads may lead to the solution of problems which are apparently of a totally different nature. A case, chosen on account of its simplicity, will serve as an illustration.

Fig. 15 (a) shows a straight stiff vertical member, with a hinged support at the bottom, supported by a horizontal spring with a modulus, S, at the top. When the load, P, reaches a critical value, Q, the structure may "buckle", as shown in Fig. 15 (b). The critical load may be determined either by direct consideration of the forces involved or by the principle of the potential energy. By the latter method one finds that the deflection in the direction of Q is equal to $\frac{u^2}{2\,l}$; then, the potential energy may be expressed as $\frac{l}{2}\,S\,u^2 - Q\,\frac{u^2}{2\,l}$. This potential energy remains zero, for any value of u, when $Q = S\,l$, which therefore, is the critical load. The same deflection, u, may be produced by the horizontal load, W, in Fig. 15 (c). If, as in Fig. 15(d), the load, P (less than Q), is introduced in addition to W, the deflection (as one finds from the condition that the potential energy is a minimum), becomes $u' = u\,\frac{Q}{Q - P}$, instead of u.

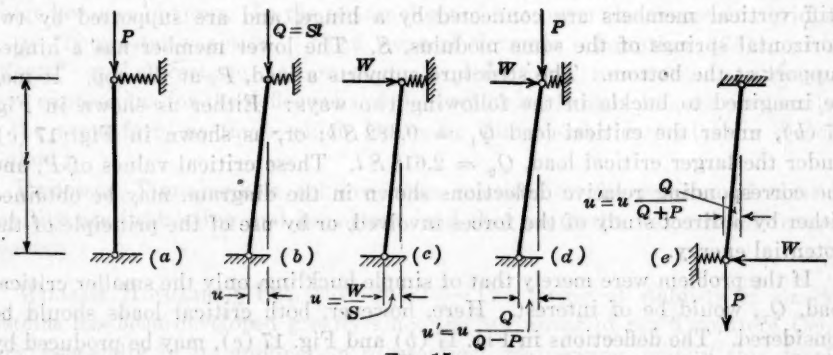

FIG. 15.

Now, turn this structure upside down, and load it as shown in Fig. 15(e). The same formula may be used, only with the sign of P reversed. The weight, P, in this case, steadies the structure, and this steadying effect is evaluated in terms of the load which produces buckling.

A steadying effect of this kind plays an important part in the mechanics of suspension bridges. This effect expresses itself in terms of the difference between what have been called the "elastic theory" and the "deflection theory".

Melan* originated the deflection theory as contrasted with the ordinary theory; the latter becomes incorrect and wasteful of material when applied to large suspension bridges. Leon S. Moisseiff,† M. Am. Soc. C. E., in discussing the suspension bridge between Philadelphia, Pa., and Camden, N. J., gave emphasis to the deflection theory, and showed its economic importance. As a matter of fact, the problem involved here may be treated from the point of view of elastic stability or buckling.

Consider the case in Fig. 16 (*a*). Turn the bridge upside down, as indicated in Fig. 16 (*b*). At a critical value of the load, P, the structure buckles by the bending of the stiffening beam. One must imagine that the sides of the polygon, which did represent the cable, remain straight, but that the polygon changes shape. By introducing this critical value in a formula similar to that applying to Fig. 15 (*e*), one may express an essential feature relating to the steadying effect which, in the case of the suspension bridge, is due to the dead load.

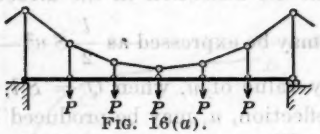

FIG. 16(*a*).

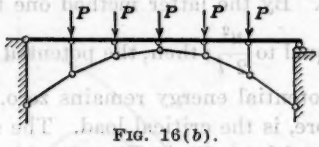

FIG. 16(*b*).

The problem referring to the suspension bridge is not quite as simple as that in Fig. 15 (*e*). The complication expresses itself through the fact that the structure in Fig. 16 (*b*) may be imagined to buckle in more than one way. The principle involved may be studied by examining a structure which is almost as simple as that in Fig. 15. Fig. 16 (*a*) shows this structure. The two stiff vertical members are connected by a hinge, and are supported by two horizontal springs of the same modulus, S. The lower member has a hinged support at the bottom. The structure supports a load, P, at the top. It may be imagined to buckle in the following two ways: Either as shown in Fig. 17 (*b*), under the critical load $Q_1 = 0.382\,S\,l$; or, as shown in Fig. 17 (*c*), under the larger critical load, $Q_2 = 2.618\,S\,l$. These critical values of P, and the corresponding relative deflections shown in the diagram, may be obtained either by a direct study of the forces involved, or by use of the principle of the potential energy.

If the problem were merely that of simple buckling, only the smaller critical load, Q_1, would be of interest. Here, however, both critical loads should be considered. The deflections in Fig. 17 (*b*) and Fig. 17 (*c*), may be produced by applying instead the horizontal forces shown in Fig. 17 (*d*) and Fig. 17 (*e*). Fig. 17 (*f*) shows a combination of the loads in Fig. 17 (*a*), Fig. 17 (*d*), and Fig. 17 (*e*). Any combination of two forces, F_1 and F_2, may be expressed by the formulas in Fig. 17 (*f*), in terms of the values, W_1 and W_2, and the corresponding values of the deflections, u_1, v_1, u_2, v_2, in Fig. 17 (*d*) and Fig. 17 (*e*), may be computed. These values, substituted in the formulas in

* J. Melan, "Eiserne Bogenbrücken und Hängebrücken," 1888.

† Leon S. Moisseiff, "The Towers, Cables, and Stiffening Trusses of the Bridge Over the Delaware River between Philadelphia and Camden," *Journal*, Franklin Inst., October, 1925.

Fig. 17 (*f*), lead to the resultant deflections, u' and v'. In these formulas the two critical values, Q_1 and Q_2, appear. With three links in the structure, and three springs, instead of two links and two springs, there would be three critical loads, and three terms in each of the expressions in Fig. 17 (*f*). With the structure turned upside down and the weight, P, hanging at the bottom, P again becomes a steadying influence, and the same formulas may be used, only with the sign of P reversed.

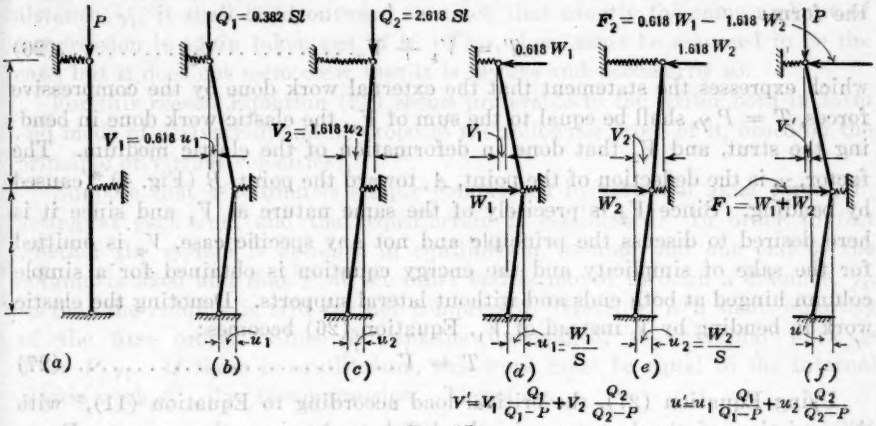

FIG. 17.

This example, of course, merely indicates that formulas of this kind apply to the suspension bridge. In a paper on buckling,* however, the speaker has given a general theory, in the light of which this application to the suspension bridge becomes obvious. In the terminology of this paper the suspension bridge is in "heterostatic action". The general formula (Equation (75)†), for "effects" in heterostatic action, such as deflections, moments, or stresses, gives expressions for the deflections of the suspension bridge that are essentially of the same form as those which Professor Timoshenko derived by a direct method.‡

Professor Timoshenko's achievements relating to the subject of elastic stability are interesting and important, and they point the way toward future research.

WILLIAM HOVGAARD,§ ESQ. (by letter).‖—The theory of stability of elastic systems has been developed greatly since the beginning of the Twentieth Century, especially in its application to practical engineering, and in this development Professor Timoshenko has taken a leading part. In this paper he has condensed the solution of a number of problems that are of practical interest to designers, not only among structural engineers, but also among

* "Buckling of Elastic Structures," *Transactions*, Am. Soc. C. E., Vol. LXXXV (1922), p. 576, especially Part IV, pp. 604–637.

† *Loc. cit.*, p. 619.

‡ "The Stiffness of Suspension Bridges," *Proceedings*, Am. Soc. C. E., May, 1928, Papers and Discussions, p. 1464.

§ Prof. of Naval Design and Constr., Mass. Inst. Tech., Cambridge, Mass.

‖ Received by the Secretary, October 5, 1927.

naval architects and others. The value of the paper is enhanced by numerous tables which facilitate the application of the formulas. Throughout all the problems the author uses the energy equation to determine the point of transition between the stable and the unstable state of equilibrium, and it is in particular on the form and derivation of this equation that the writer wishes to comment.

In Article III, the fundamental equation of energy, Equation (9),* is given the form:

$$T = V_1 + V_2 \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (26)$$

which expresses the statement that the external work done by the compressive forces, $T = P\,\gamma$, shall be equal to the sum of V_1, the elastic work done in bending the strut, and V_2 that done in deformation of the elastic medium. The factor, γ, is the deflection of the point, A, toward the point, B (Fig. 4),* caused by bending. Since V_2 is precisely of the same nature as V_1 and since it is here desired to discuss the principle and not any specific case, V_2 is omitted for the sake of simplicity and the energy equation is obtained for a simple column hinged at both ends and without lateral supports. Denoting the elastic work in bending by V instead of V_1, Equation (26) becomes:

$$T = V \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (27)$$

Using Equation (27), the critical load according to Equation (11),* with the omission of the last term in the brackets, becomes the same as Equation (1).†

Another formula which has been used in connection with the problem of the buckling of plates, is obtained by assuming that the work done by the compressive forces is zero, but that the plate buckles sidewise by a small amount. Thus, there will be introduced an additional potential energy of bending, V, and at the same time the potential energy of compression decreases by an amount, V_1. Hence, if the potential energy of the system is to remain constant, the equation of energy becomes:

$$V - V_1 = 0 \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (28)$$

Since both Equations (27) and (28) give the correct result they must be equivalent to each other; but this does not seem immediately obvious. Imagine first that a simple compression displacement, γ_1, takes place in a pin-ended column without any bending, the external work being T, while, at the same time, an internal work, V_c, is done by the elastic compression of the column. Since the total axial stress force in the column is equal to P, $V_c = P\,\gamma_1 = T$. Assume that the column is deflected a very short distance sidewise in bending, so that its axis is lengthened by an increment, γ_2, and a certain amount of energy, V, is absorbed in bending, while, at the same time, an amount of energy, V_1, is released due to the elongation. Equating the external and the internal work:

$$T = V_c + V - V_1 \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (29)$$

* *Proceedings*, Am. Soc. C. E., April, 1929, Papers and Discussions, p. 859.
† *Loc. cit.*, p. 856.

Since V_c is equal to T, Equation (29) may be written, $V - V_1 = 0$, which is the same as Equation (28).

It is thus shown that Equation (29) is equivalent to Equation (28); but it is not quite obvious that it is also always equivalent to Equation (27). Evidently, this will be true only if $V_c = V_1$; that is, if γ_1, the deflection of P, is always equal to the elongation, γ_2, because, in that case, Equation (29) reduces to, $T = V$. In other words, after the column has been compressed a certain distance, γ_1, it shall bend outward so much that exactly the same amount of compression is again taken out of it. This, then, must be assumed to be the case, but it does not seem clear that it is always and necessarily so.

For this reason Equation (28) seems preferable to the writer both in form and mode of derivation, and he proposes the following proof of it, based on the principle of virtual velocities:

Suppose that a column is simply compressed by terminal axial forces, P, acting at each end, and that equilibrium is established. In order to test whether the system is actually in equilibrium, assume that one end of the column is fixed and that P at the other end is moved through a distance, γ_1, so that the column is still further compressed. Here, γ_1 is a small quantity of the first order. Since P remains unchanged, the external work is $T = P\,\gamma_1$. If there is equilibrium, this work must be equal to the internal elastic work, V_c, due to compression. Therefore,

$$T = V_c \qquad (30)$$

In order to determine whether the equilibrium is stable, it is necessary to consider quantities of the second order. The column is still in the compressed state, but both ends are now regarded as fixed and the column is given a sidewise virtual deflection. This deflection is a small quantity of the first order, but the elongation and the bending strains produced thereby are of the second order. A certain amount of energy, V, is absorbed in bending and an amount, V_1, is released due to the elongation caused by bending, but both these quantities are of the second order. Since no external work is done in this case, the condition necessary for stable equilibrium is that $V > V_1$ and if the column is in the state of transition between stable and unstable equilibrium, $V = V_1$, as given in Equation (28). This reasoning is consistent with Equation (29) if in that equation T is made equal to V_c and these quantities are regarded as being of the first order, while V and V_1 are of the second order. The stability of rectangular plates under edge thrust or edge shear is a problem of considerable interest to naval architects, who have not yet made full use of the results achieved recently in this field of the theory of elasticity. The application of the formulas to ship-plating is complicated, however, by the indefinite mode in which the plates are supported by frames and longitudinals. Furthermore, the presence of riveted seams or butts and the transverse load due to the external water pressures, complicate the problem.

There are at least three cases in which buckling should be considered in the strength of ships. First, when the outer shell or the deck plating is compressed under longitudinal bending of the entire hull; second, when the

outer shell plating on the sides is subject to shearing due to forces acting on the entire hull; and, third, when parts of the structure are exposed to excessive local forces, as may occur, for instance, in docking or grounding.

The two first cases are of the greatest importance in lightly built, high-speed vessels, such as fast cruisers and torpedo-boats. In such vessels, the scantlings of the outside plating and of the strength deck are determined primarily with regard to the stresses in longitudinal bending. Generally, a certain maximum calculated stress is allowed or aimed at in the design, but in considering the corresponding factor of safety the strength in buckling or wrinkling of the plating is not generally taken into account. Nor is the choice of spacing of the frames or longitudinals ordinarily influenced by considerations of buckling.

Example 1.—Consider the bottom plating of a torpedo-boat in which the transverse frame spacing, a, is 21 in. and the thickness of the plating, t, is 0.25 in. The spacing, h, of the longitudinal girders is 78 in., and it is desired to examine the effect of buckling pressure when the bottom plating is under longitudinal compression, as in the hogging condition. In a panel enclosed between two adjacent frames and two adjacent longitudinals all the edges are regarded as simply supported. Then, $\frac{h}{a} = \frac{78}{21} = 3.71$; $E = 30 \times 10^6$ lb. per sq. in.; $\sigma = 0.3$; and the flexural rigidity is:

$$D = \frac{E\,t^3}{12\,(1-\sigma^2)} = \frac{30 \times 10^6\,(0.25)^3}{12 \times 0.91} = 42\,900$$

The critical stress is obtained by means of Equation (31):*

$$p_{cr} = \left(\frac{h}{a} + \frac{a}{h}\right)^2 \frac{\pi^2\,D}{t\,h^2} = \alpha_1 \frac{\pi^2\,D}{t\,h^2} \quad \ldots\ldots\ldots\ldots\ldots\ldots (31)$$

Making the proper substitutions, the value is, approximately,

$$p_{cr} = 15.8 \times \frac{9.87 \times 42\,900}{0.25 \times 6\,084} = 4\,440 \text{ lb. per sq. in.} = 2 \text{ tons per sq. in.}$$

In vessels of this class it is usual to allow a permissible designed stress in compression of about 6 tons per sq. in. in the extreme fibers, corresponding to perhaps 4 to 5 tons per sq. in. in the strakes of plating here under consideration. Hence, it appears that buckling would be very likely to happen, but, although there are cases on record where buckling resulting in permanent strains has occurred, on the whole, it must be regarded as an exception. Another consideration is that elastic buckling may take place quite commonly in vessels of this type without being noticed because the deformation is arrested by adjacent longitudinal members before over-strain occurs, and when the stresses cease, the plating goes back to its original position. The effect in this case is that certain plates, which form part of the flanges of the ship girder, shirk their duty in compression and greater longitudinal stresses are thrown on the remaining strakes. Such incipient buckling has been previously described by the writer.†

* "Applied Elasticity," by S. Timoshenko and J. M. Lessells, p. 292.

† "Structural Design of Warships." E. and F. N. Spon, Lond., 1915, p. 27.

Basing conclusions on various tests and on the practice common in bridge construction, it was estimated that the limiting spacing of longitudinal stiffeners for which the plating may be assumed to be fully effective in compression, should be equal to about eighty times the thickness. If this spacing were exceeded it was recommended to reckon as effective only a band of plating directly attached to the longitudinal stiffeners of a width equal to fifty times the thickness, that is, a strip of width, 25 t, on each side. In the torpedo-boat discussed in Example 1, the ratio, h to t, is more than 300, but in battleships it may be as small as 80 or less, especially in the strength deck.

It must be borne in mind also that in Equation (31), all the edges are assumed to be simply supported, that is, $w = 0$ and $\frac{\sigma^2 w}{\sigma x^2} = 0$; but whether the last condition is fulfilled seems doubtful. Probably there is considerable clamping effect along all four edges and in such cases buckling will not occur until a greater stress has been reached.

As regards buckling or wrinkling by shearing the writer has heard of no case in which this has occurred due to the general shearing of the entire hull of a ship. If it did occur, it would probably take the form of wrinkling between transverse frames, but although permanent damage of this kind may not have been observed it is still possible that very light elastic wrinkling may have taken place unnoticed, disappearing as soon as the shearing forces cease to act. In cases where the shearing stresses actually have been excessive, relief may have been obtained by local shearing of rivets in the horizontal seams, resulting in greater stresses being thrown on the flanges of the ship girder. Such shearing of the rivets in the longitudinal seams in the region of the neutral axis is not uncommon and has led to the adoption of triple-riveted seams in certain classes of ships in the region most exposed to shear stresses.

Example 2.—It is of interest to calculate the buckling stress in shear for the same panel as considered in Example 1. From Table 9,* α_1 will be equal to about 6.0 and since all the other elements are the same as in case of buckling by compression, in which $\alpha_1 = 15.8$, the critical buckling stress in shear is,

$$p_{sr} = \frac{6.0}{15.8} \times 4\,400 = 1\,670 \text{ lb. per sq. in.} = \tfrac{3}{4} \text{ ton per sq. in.}$$

Actually, the maximum calculated shearing stress may reach about 4 tons per sq. in., so that buckling of the plating would seem very likely to occur; but, again, the boundary conditions are probably more favorable than was assumed.

Local buckling and wrinkling is known to have occurred in the docking of ships, both in floor-plates and in the lower parts of bulkheads when excessive pressures have existed on certain docking blocks. The stresses are not readily calculated in such cases on account of the complexity of the structure; furthermore, the pressure on the blocks is difficult to determine. The writer believes that naval architects can learn a great deal from civil engineers, but he believes also that civil engineers could obtain much valuable information

* *Proceedings*, Am. Soc. C. E., April, 1929, Papers and Discussions, p. 869.

by an analysis of shipbuilding practice. Evidently, civil engineering lends itself much better to the application of scientific methods than naval architecture. In ships, the straining actions are far more uncertain and varied in character than in stationary structures, and dynamic forces of great magnitude occur quite frequently.

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PAPERS AND DISCUSSIONS

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THE SURVEYING AND MAPPING ACTIVITIES OF THE FEDERAL GOVERNMENT

Discussion*

BY MESSRS. WILLIAM BOWIE, WILLIAM O. TUFTS, HENRY J. SHERMAN, RUFUS H. SARGENT, AND ARTHUR H. KIDDER.

WILLIAM BOWIE,† M. AM. SOC. C. E. (by letter).‡—The author has performed a valuable public service in setting forth detailed information as to which agencies of the Government have been making surveys and maps, and how, under the Board of Surveys and Maps, the work of the several Government agencies is co-ordinated.

As Mr. Norcross has shown,§ there are a number of Government organizations which carry on surveying and mapping activities merely as an incident to engineering operations or in the administration of some Federal law. There are, however, several Government agencies which have been charged by Congress with the surveying and mapping of the United States. It is to these organizations, principally the U. S. Coast and Geodetic Survey and the U. S. Geological Survey, that the public looks for those surveying and mapping data which are so essential in carrying on many engineering, industrial, and commercial undertakings. Only a little more than 40% of the area of the United States is covered by the topographic map and at least one-half the covered area should have a revision of the maps in order to bring them up to date. If one should look at the index map of the topographic sheets of the Geological Survey, he would find that very few States of the Union have been completely covered. The quadrangle sheets are scattered widely, many of them having been made for some special purpose. As a result, for any large area, complete topographic data are unavailable for such large undertakings as the

* This discussion (of the paper by T. W. Norcross, Assoc. M. Am. Soc. C. E., presented at the meeting of the Surveying and Mapping Division, Washington, D. C., April 26, 1928, and published in April, 1929, *Proceedings*), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

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‡ Received by the Secretary, May 15, 1929.

§ *Proceedings*, Am. Soc. C. E., April, 1929, Papers and Discussions, p. 877.

development and extension of a highway system and the study and planning necessary for flood control of rivers.

As a part of the conservation of natural resources, it would seem most desirable that the mapping of the country should be completed at an early date. It is only with a knowledge of the terrain, such as is furnished by the topographic map, that one can discover what the natural resources are, where they are located, and whether they can be economically developed and conserved. The United States is far behind other progressive nations in regard to its topographic mapping.

The situation as to the control surveys of the country, on which the topographic maps are based, is probably in better condition than the topographic mapping. Since the beginning of the Coast and Geodetic Survey, under Ferdinand Hassler, its first Superintendent, in 1816, first-order triangulation has been executed, first, with a view to controlling the charts of the coast and, later, the surveys and maps of the interior of the country. More than one-half of the first-order triangulation has been completed and even the work done under Hassler is of standard accuracy and will not have to be reproduced. Much more triangulation should be done, but the major part of that remaining to be executed consists of second-order work which will control the areas surrounded by arcs of first-order triangulation.

There are now (1929) nearly 60 000 miles of first-order leveling in the United States, most of which has been executed by the U. S. Coast and Geodetic Survey. Other organizations which have executed leveling are the U. S. Geological Survey, the Mississippi River Commission, the U. S. Lake Survey, and several railroads There are a few lines of this net which have been classed as "first-order" which perhaps should be classed as "second-order". The completion of the level net, composed of first and second-order leveling, will require approximately 70 000 additional miles of leveling.

It has been estimated that approximately $5 000 000 will be needed to pay the cost of completing the first and second-order triangulation and leveling nets of the country, and this expenditure should be made by the Federal Government. When these nets are finished, no place will be more than about 25 miles from a fundamental bench-mark or triangulation station. Intervening areas will be covered by third-order horizontal and vertical control, by the States, counties, the topographic branch of the Geological Survey, and by private corporations and individuals. Necessarily, the Federal Government should not be required or expected to make detailed control surveys of local areas (unless they are Federal areas, such as National Parks and Forests and Indian Reservations) except in co-operative topographic mapping.

Mr. Norcross has conveyed a very clear idea as to how the surveying and mapping of the Government is being carried on and how they are co-ordinated. The accuracy of the Governmental work appears to be all that is required for general purposes. Can as much be said for the co-ordination of the surveying and mapping of the cities, States, counties, and of private corporations and individuals? The writer believes that the answer must be decidedly negative. There is practically no co-ordination of effort by agencies or individuals outside the Federal Government, but this condition will probably be greatly improved

in the years to come. Surveying and mapping are the basis for many engineering operations, and the maps are essential in carrying on numerous lines of industrial and commercial activity, other than engineering. It is the hope of those who are devoting their entire time to surveying and mapping that more thought will be given to these subjects in the interests of the elimination of waste in industry.

WILLIAM O. TUFTS,* ESQ.—This subject is most interesting and valuable, especially to engineers at a distance from Washington, D. C. Even those resident in the Capital City often are unacquainted with certain surveying and mapping activities of importance. As the Federal program becomes more complete and effective the resulting maps will serve the map users of the country to better advantage, and commercial maps will be improved to a similar degree.

It is quite evident that the engineers of the United States have not been co-operating actively in seeking to advance the map-making program of the Federal Government. The map makers have been obliged to present the facts in the case to those responsible for appropriations and in this they have been subject to a criticism implying that they were being influenced by a desire to extend their own bureau activities and make their own job more important. The map users of the country, and this refers particularly to the engineers, should make themselves heard in demanding the completion of the topographic mapping of the United States within the next few years. This program will not materialize unless the map users are willing to show that accurate maps are a vital necessity for the development of National resources.

HENRY J. SHERMAN,† M. AM. SOC. C. E.—The question concerning the co-ordination of basic data is pertinent. It is presumed that the work of the U. S. Coast and Geodetic Survey on Tar River in North Carolina, where the elevations were referred to mean tide level, was done for the purpose of determining the amount of dredging below the low-water plane. Consequently, tide-gauges were set at suitable intervals along that stream and a series of readings taken to establish the low-water plane. If there were no Coast and Geodetic Survey bench-marks referred to mean sea level within a reasonable distance of the river (say, two miles), there is some excuse for not referencing the low-water plane with mean sea level, but the situation is different in the Eastern States where the Sandy Hook Datum is available at many convenient points. The speaker's experience is largely with the Delaware River and its tributaries for which those elevations may be obtained within a short distance of that river, and yet it is believed this co-ordination has never been done.

Here is an opportunity for the Board of Surveys to use its good offices to establish a practice which will benefit the profession generally and reference permanently the working datum for river and harbor improvements.

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RUFUS H. SARGENT,* ESQ.—Few people realize, first, that Alaska has approximately one-fifth the area of the United States, namely, 586 400 sq. miles; and, second, that the U. S. Geological Survey has mapped approximately 45% of it. Other agencies have mapped a small additional area. All this is exclusive of the U. S. Coast and Geodetic Survey which has done an excellent work along the coast.

ARTHUR D. KIDDER,† M. AM. SOC. C. E.—From the viewpoint of the man in the ranks, the speaker may be able to present the ideas of those who are employed to use the instruments, methods, and plan of organization provided by the Federal Government.

The subject of Government surveying activities is a large one. There exists an erroneous notion in regard to the public land sub-divisional surveys. It should be more generally observed that the same effort has been made to modernize the work of boundary surveys by the Federal Government that is found in other lines of engineering activities. About 1900 an attempt was made to overcome the use of the magnetic needle in locating boundaries. The improvement was accomplished in Government surveys because authority to direct the methods was vested in the administrative officers; but the magnetic needle is still used to a very large extent in the running of private land boundaries, where title has passed from the Government. The solar transit instrument, needed to modernize the method, was developed by surveyors engaged on Federal work.

Necessarily, the sub-division of the public lands has had to proceed much in advance of high order control. Public land sub-divisions were largely completed long before precise position in latitude and longitude was known. However, realizing the great need for co-ordination, the General Land Office has found it profitable in recent years to detail a cadastral engineer to accompany the triangulation parties of the Coast and Geodetic Survey, so that points in the public land sub-divisional system could be connected to the triangulation work. This has been accomplished at a negligible cost, and much valuable data have been accumulated concerning the geographic position of the sub-divisional surveys, heretofore unknown.

It has also been necessary to improve the making of township plats. The older ones were deficient in map data, but the addition of the map features resulted in burdening the plat to the disadvantage of the base. There is a sharp line of demarcation regarding what is appropriate to show on a plat, so as to give prominence to the sub-divisional characteristics and to avoid that which belongs strictly to the topographic map.

An excellent example of combining the sub-divisional survey with the topographic survey is found in the lands of the five civilized tribes of Indian Territory. A similar combination has been successful in other Indian reservations, and in Alaska, where large areas were to be subdivided; but from the standpoint of economic production of routine work it is now conceded that it is not best to combine the topographic survey with the land sub-divisions.

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The report* of the Committee on Boundary Surveys did not answer some questions which will naturally arise in the minds of those who are engaged in this class of work. It gives the impression that the Committee will compile a manual for the subdivision of the public lands. It is doubtful if that is what is intended. Presumably most members are more interested in the boundaries of lands that are in private ownership. The sub-division of the remaining public lands is an important one, but it is in the hands of engineers who have specialized in that class of work. It appears obvious that the work of the Committee will be more profitable if directed to the improvement of the survey of private land boundaries. Comparatively few trained engineers are engaged in that kind of work, which is done mostly by county surveyors, many of whom are unacquainted with modern standards.

Another point in the report of the Committee is in regard to the matter of riparian rights of land owners.† Apparently, it is intended to revise the old scheme of riparian rights, although it hardly seems legally possible to do so. The work of the Committee will be better directed if a study is made of the existing laws. Riparian rights are based on the laws of the forty-eight States. These laws differ, but they fall into only a few classifications as to groups. Riparian rights are established by law and cannot be changed by the engineer. The engineer must study the laws.

The Red River boundary survey grew out of a dispute over the location of the line between the States of Texas and Oklahoma, where it follows the course of Red River. It dates back to the days when much of the region of the Southwest was Spanish territory. There were treaties from time to time between the Governments of the United States, the Republic of Texas, and the Spanish crown, regarding this boundary. Provision was made a number of times for a survey, but nothing was accomplished jointly by the jurisdictions involved. In 1919, oil was discovered in the bed of Red River, which caused much confusion over the location of the boundary between the States. The dispute was carried to the Supreme Court of the United States, which was called upon to interpret the treaties giving definitions to the boundary, and then to apply the terms to the location on the ground.

The Court held that the boundary is along the south bank of the river, at an elevation half way between mean low water and mean high water. The problem of locating the boundary, especially through the oil field, involved precise control for geographic position, precise levels, and a retracement of the sub-divisional surveys on both sides of the river.

Red River, in its lower 90 or 100 miles, where it constitutes the boundary, has a very active habit of changing its banks, cutting on one side and building on the opposite. The work of erosion and accretion is constantly going on, and there are frequent avulsive cut-offs. The land sub-divisions on the Texas side date back to 1830, while those on the Oklahoma side were made in 1898. Owing to the many changes in the intervening years, some parts of the river valley were subdivided in both jurisdictions—in Texas in 1830, and in Indian Territory in 1898.

* *Proceedings*, Am. Soc. C. E., April, 1929, Papers and Discussions, p. 899.
† *Loc. cit.*, p. 907.

In some places the river had worked from north to south, and the intervening area had not been subdivided in either jurisdiction. Many places were identified, where large bodies of land had been cut off from one side, without the destruction of the land, and had become attached to the opposite mainland. In such cases the boundary remained where it had been before the avulsive cut-off occurred, but in the case of erosion or accretion the boundary followed the changes. The engineer engaged in the survey of such a boundary must necessarily observe all rights; he cannot change the law to suit a convenience.

A great deal of benefit is gained from the point of view of the man in the ranks. It is always an inspiration to observe the work of the specialist in another field. All cannot be experts in all subjects. Particularly in the surveying work of the Federal Government better results are obtained by a division of the work into the major classes, geodetic, hydrographic, topographic, and cadastral.

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GOOD MAPS ARE A SOUND INVESTMENT

Discussion*

By Messrs. E. K. Smith, D. L. Reaburn, Francois E. Matthes, W. E. Parker, Hugh C. Mitchell, and Dr. Helen M. Strong.

E. K. Smith,† Assoc. M. Am. Soc. C. E. (by letter).‡—It is certain that engineers in the United States, especially those in the Government service, recognize the great value of good maps. In fact, the Temple Bill and the support which has been given it by organized groups of engineers is evidence of this. It is equally evident that the average engineer has not realized the necessity for "selling", or does not feel that he is in a position to "sell", this same idea to the general public. This paper deserves widespread presentation to the people at large to acquaint them with the simple every-day business value of Government maps.

Unquestionably, European countries are far better supplied with complete maps than the United States. These countries are smaller, it is true, yet the United States is better equipped in many respects for the rapid completion of such an undertaking. It is hoped that Major Lohr's paper will furnish additional inspiration and interest to further the completion of triangulation, secondary control, and the detail topographic work of mapping the United States. From an ordinary business standpoint this work is just as beneficial in its own degree as reclamation projects, or the Muscle Shoals Dam, for which large sums of money are appropriated. It is only a matter of explaining the needs and uses of these maps to the public at large and to the members of Congress.

D. L. Reaburn,§ M. Am. Soc. C. E. (by letter).‖—This paper sets forth completely the value of topographic maps and the many uses to which they

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‡ Received by the Secretary, May 8, 1929.

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‖ Received by the Secretary, June 26, 1929.

may be put and emphasizes the importance of accurate topographic maps to an engineer engaged in the location and design of a project. The larger part of the topographic mapping in the United States has been done on a scale of 1:62 500, or smaller. It is impossible to show on such maps all the topographic details necessary for many engineering studies. The cost of large scale detail maps is not justified except in thickly settled sections; or for the special purpose of an engineering project, where it is economically possible to make maps of flat and rolling country on scales of 2 000 ft. to the inch, or larger, with 2 and 5-ft. contour intervals and with sufficient accuracy for the location and design of many projects—for example, the map of Los Angeles County, California, now being made by the U. S. Geological Survey.

Engineers engaged upon the location and design of a project, often criticize and condemn a U. S. Geological Survey map because it is not accurate enough. This is because they try to use it for something for which it was not intended.

One should bear in mind that the contours on these maps are sketched with a degree of accuracy consistent only with the scale, and that a small difference in horizontal position may correspond to a considerable difference in elevation. When properly used these maps furnish all the necessary information for the selection of general routes for railroads, highways, aqueducts, and other engineering projects.

They take the place of reconnaissance surveys and reduce to a minimum the number of detail location surveys required to select the best route. In many cases, the alignment can be determined directly from the maps without making detail route surveys.

The U. S. Geological Survey maps, where available, were used to great advantage in the location of the Los Angeles Aqueduct. In one particular section, where no maps existed, the maps, afterward made, indicated a better route than that over which the aqueduct was built. In connection with studies for the selection of the route for the proposed Colorado River Aqueduct, the Department of Water and Power of Los Angeles has made topographic maps and a relief map from which comparative estimates of cost are found.

FRANCOIS E. MATTHES,* ASSOC. M. AM. SOC. C. E.—The author has rendered the Engineering Profession a distinct service by stressing the basic importance of good maps in connection with the manifold projects and enterprises of modern industrial civilization. He is right in stating† that if the economic value of maps is not sufficiently recognized to-day it is because map makers as a rule occupy themselves too exclusively with the technical phases of their work, and forget to make adequately understood the purposes that maps serve and the economies that are effected by their intelligent use. All work of a highly technical and specialized nature needs to be skillfully interpreted and explained to the laity in order that it shall receive due recognition and support.

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† *Proceedings*, Am. Soc. C. E., April, 1929, Papers and Discussions, p. 889.

The speaker is gratified to note that Major Lohr did not omit mention of the part that topographic maps play in geologic investigations. He referred* in particular to the value of such maps in connection with the study of oil-producing areas. The speaker has seen detailed maps of a well-known oil field in California on which practically the entire dome structure of the oil-bearing strata stood clearly revealed in the contouring. Even the minor folds in the strata and the faults were discernible to the trained eye. Of course, in most oil fields, the dome structure is only partly expressed in the topography; in some fields it is not expressed at all; but in every case the oil geologist needs an accurate topographic map in order to determine the underground structure.

What is true in oil geology is true also in mining geology, in fact, in almost every phase of economic geology. The economic geologist of to-day must not only have an understanding of the complex geologic structures with which the ore bodies or other deposits to be developed are associated, but he must determine the details quantitatively, and this, manifestly, he cannot do without a good topographic base.

More important, even, and more difficult to fill are the needs, in the way of maps, of those who are engaged in pure geologic research. These scientists undertake to discover the fundamental facts concerning the make-up of the earth's crust, the origin, arrangement, and the age of rock formations; they supply those basic data of which the economic geologist makes practical use in developing the mineral resources of the country. Necessarily, this fundamental research must be carried over large areas, ultimately over the entire National domain, and, consequently, it demands extensive mapping on a unified plan, both for horizontal and vertical control. This fact was duly recognized in 1879 when the Government of the United States incorporated its principal map-making agency with the Geological Survey instead of with the War Department, contrary to the practice of those European nations which by stress of circumstances have felt themselves compelled to subordinate their industrial development to the exigencies of military preparedness.

The speaker's work in recent years has been largely along one line of geologic research, that known as physiography, or perhaps more properly, as geomorphology. His has been the fascinating task of tracing the successive steps in the evolution of river valleys and glacial canyons, in the progressive upbuilding and the progressive destruction of entire mountain ranges and plateaux. A large share of the evidence is found in topographic forms. Every summit, spur, canyon, valley, ravine, every escarpment, bluff, terrace, every change in declivity on mountain slope or in valley floor, gives its bit of testimony. It follows that the geomorphologist is peculiarly dependent upon topographic maps, and that to serve him adequately they must be quantitatively accurate as well as qualitatively expressive. A single erroneous elevation, the omission of a "wiggle" in a contour line, sometimes suffices to mislead him. Those who have not had experience in this type of research may find it difficult to realize what a calamity it is when in the midst of an

* *Proceedings*, Am. Soc. C. E., April, 1929, Papers and Discussions, p. 896.

elaborate study of possibly far-reaching import one finds that in a critical spot the map cannot be trusted; on the other hand, what a satisfaction it is to find the map reliable throughout and to spell out with its aid a consistent sequence of events in earth history. It needs hardly to be added that the geomorphologist, of all map users, is willing to agree with Major Lohr that good maps are a sound investment, with the emphasis on "good".

W. E. Parker,* M. Am. Soc. C. E.—The author mentions† the importance of having accurate maps and sounds a word of warning that the users of maps should test them for accuracy. Unfortunately, the users of nautical charts can not test them and so must accept them as accurate. Charts that bear the stamp of the Government are usually accepted as accurate in every detail, and that perhaps accounts for many of the wrecks that have occurred along charted coasts. Until recently, it has not been possible, except at very great expense, to make a hydrographic survey that is accurate in all particulars where the bottom is rocky or where there are pinnacles, boulders, or coral heads. The problem that has faced the U. S. Coast and Geodetic Survey since the earliest days has been so vast that it has not been possible always to survey with that degree of detail that would insure the detection of all dangers to navigation. Only the most important parts of the coast have been surveyed thoroughly and these at very great cost. Consequently, the entire work is not adequately done.

The fact that a coast has been charted is no indication that it has been thoroughly surveyed. Cartographers construct charts from such data as may be available: Good surveys, indifferent surveys, and even from reconnaissance sketches, so that the chart user has no way of knowing, if he has a chart in his hand, whether it is good, bad, or indifferent. It may be said that the entire coast of the United States (exclusive of Alaska) is charted, which is true, but the point of interest is as to how much of it is charted adequately for the needs of commerce to-day.

Many of the charts of the New England coast are based upon very old surveys which were executed by methods and with appliances which now would not be tolerated. Along the Southern Atlantic and Gulf coasts, important changes have taken place since the surveys were made on which the charts of those waters are based. Many of these charts, therefore, are either incomplete or they represent conditions that do not exist to-day (1928). They are not adequate for present-day navigation.

Hugh C. Mitchell,‡ Esq.—The speaker wishes first to emphasize Captain Lohr's plea§ for fitting the map to the needs of the user, and then to show from a limited experience that "engineers are sold on maps" only in a general way. He admits that engineers are sold on maps, and that they secure the best that can be obtained easily. Unfortunately, these are often of poor quality and just do serve the purpose in hand, when really good maps would

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† *Proceedings*, Am. Soc. C. E., April, 1929, Papers and Discussions, p. 890.

‡ With the U. S. Coast and Geodetic Survey, Washington, D. C.

§ *Proceedings*, Am. Soc. C. E., April, 1929, Papers and Discussions, p. 888, *et seq.*

not only have reduced the cost but would have made better plans possible. Really good maps are not so plentiful, however, and many an engineer is without knowledge or experience relating to them. He, therefore, goes ahead like the New Guinea pigmy hewing trees with his stone axe, and "getting away with it", because the stone axe is the only kind he knows.

In order to have the type of map best suited to his work the engineer often needs education both in map-making and in map using; he must either select his maps from those offered and be able to know whether they will give him all he needs, or he may find it necessary to construct his maps himself. If he does his own mapping he is prone to fall into another rather common error, that of giving greatest thought to first cost. This is a proper thing to do only when tempered with vision and a knowledge of the large dividends paid in the form of savings by securing the best type of maps that can be produced for the project under plan.

The speaker planned and executed the 1912 topographic survey of Cincinnati, Ohio. After a reconnaissance for what might be termed a precise topographic map, he was one day called into conference with the Consulting Engineer, a man of international reputation in the design of sewers and disposal works, and was shown a map of another city which had been used for just such work as was being started in Cincinnati. The engineer in charge of the sewerage investigation wanted the more precise type of map, but the Consulting Engineer was hesitating because of the cost, and he asked the speaker what criticism he had to make of the map spread on the table before them. It was a good map of its type, perfectly satisfactory for general planning, but absolutely unsuited in scale or accuracy for detail studies.

Consequently, a more precise map was made, which was controlled by first-order triangulation and levels, with an accuracy of plotting of contours and culture that would permit the draftsman in the office to take from any part of it cross-sections of an accuracy comparable with what could be obtained in the field with transit and stadia. As the map grew, there came a change in the attitude of many of the engineers of the organization to whom such a map was a new experience. It was used more and more for detail planning, not only by the sewer engineers, but by many others. Almost every Sunday, the active sheets that were in the field six working days of the week, were requisitioned by some one, either the park engineers, who wanted tracings of the contours (which were still in pencil) of certain completed parts representing areas where park improvements were in progress; by the bridge engineers who were seeking the most advantageous location for a viaduct across some ravine; or by the engineers planning an interurban traction loop. Even the sewer engineers themselves, who at first wanted only a map for general studies, for the computation of run-off and drainage areas, etc., began making more and more detailed use of the map, until they placed upon it almost more of a load than it could stand. They had always been "sold" on maps, but many of them had never before had a chance to use an accurate one of the type best suited to their work, and now with the chance before them, they found in this accurate map "a new tool with a sharp edge" which they wanted

to use to the utmost. The New Guinea pigmy uses his stone axe to hew his trees, but the engineer, using a really accurate map for the first time, finds such joy in this new and most powerful tool that he uses it not only to hew his trees, but also to sharpen his pencils, which is just another way of stating that the engineer still needs to learn a great deal about maps—how they are produced, what they cost, and how they may be used to show large savings in dollars and cents.

DR. HELEN M. STRONG.*—Captain Lohr has discussed an important question, the significance of which extends into the commercial map field. Good maps of various kinds, including topographic maps, are of great significance to the general business man. Inquiries received in the Department of Commerce indicate a desire on the part of many to use maps, but there is lacking, in general, a knowledge of what maps are available and the way in which they may be used. A map puts into one picture many aspects of domestic and foreign trade problems, revealing inter-relations, and suggesting advantageous business operations.

When business men utilize, more freely, commercial and other maps relating to their problems, they will not only save themselves losses but will operate with increased efficiency, and will be in a position to take advantage of many more strategic opportunities. Maps are as essential to the exporter and distributor as they are to the engineer.

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BOUNDARY, TOPOGRAPHIC, LOCATION AND CONSTRUCTION, AND CONTROL SURVEYS

PROGRESS REPORTS OF THE COMMITTEES OF THE SURVEYING AND MAPPING DIVISION

Discussion*

BY MESSRS. EDWARD B. HILL, C. O. SHERRILL, HENRY J. SHERMAN, WALLACE H. HALSEY, J. K. FINCH, R. H. RANDALL, CHARLES B. STANTON, CARROLL F. MERRIAM, WARREN C. TAYLOR, M. W. WEIR, AND PAUL H. UNDERWOOD.

EDWARD B. HILL,† ASSOC. M. AM. SOC. C. E. (by letter).‡—The report on topographic surveys is timely and interesting. It was a pleasure to note that Mr. Brown included the subject of aerial mapping. This is an interesting subject the practicability of which has already been demonstrated by many engineers.

The writer compiled a map from 1 260 vertical aerial photographs covering about 650 miles of the Mississippi River from St. Paul, Minn., to the mouth of the Missouri River. In this work, the single lens pictures were used with marked success. They were assembled on the basis of the control established by the Mississippi River Commission in 1890 and more recent traverse points of the U. S. Geological Survey. The average scale of the photographs was 1:13 200, and this was used in the compilation, but the completed map will be published on a more common scale.

Naturally, during thirty-nine years many important changes have occurred in the river. In order to make a comparison, a small section of the map compiled from the aerial photographs was reduced to the same scale as the map made by the Mississippi River Commission in 1890. It was interesting to note that these maps coincided not only in the larger features, but also in many of the finest details. Experience in this and other fields of work has convinced the

* This discussion (of the Progress Reports of the Committees of the Surveying and Mapping Division on Boundary, Topographic, Location and Construction, and Control Surveys, presented at the meeting of the Surveying and Mapping Division, Washington, D. C., April 26, 1928, and published in April, 1929, *Proceedings*), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

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‡ Received by the Secretary, April 5, 1929.

writer that sufficient ground control is the most essential factor. Given adequate control in regions of moderate relief, maps as accurate as those made by ground surveys can be constructed from either single lens or tri-lens photographs.

It seems likely that as time goes on the colleges and the Engineering Profession as a whole will appreciate this new method of mapping and will use it more extensively.

C. O. SHERRILL,* M. AM. SOC. C. E. (by letter).†—A most important contribution to the science of surveying will be made by the Committee of the Surveying and Mapping Division on Topographic Surveys if it can secure an understanding of the requirements of topographic surveys for the different uses to which they are to be applied.

Frequently, a great waste of time, effort, and money results from topographical surveys being made in more detail than is required by the object to be attained. Not only is there a general lack of standards as to scales on maps, but this is equally true as to contour intervals used on the different scales.

On all its geographical and military surveys the Government has long ago discovered the important relation which exists between contour interval and scale; that is, the map distance representing different slopes. As a result of such standards, these Government maps are easily comprehended, or, in other words, the reading of these maps becomes a very simple matter due to this definite relation between scales, contour intervals, and slopes.

In no field is there such constant use of topographical maps as in that pertaining to the art of war. From the General to the "Buck Private" in the rear rank, every man must be able to locate his position and to determine distances, slopes, and elevations from his map. During the World War, the failure on the part of troops, only a short time in service, to appreciate this importance led to many deaths on the battlefields in all the Allied and German armies, due to artillery fire by friendly troops.

Facility in the use of the maps is accordingly a prime consideration. Standardization of scales, contour intervals, and map distances, as well as many other considerations of a practical nature, are also of vital importance. For instance, it is foolish to undertake to place 1-ft. contours on a map of such small scale that within the limitations of the draftsman it is impossible to space these contours so as to represent accurately the ground being portrayed by the map. On the other hand, it is a needless waste of time to make any topographical map of a scale the least bit larger than is necessary to allow the essential horizontal details to be shown with accuracy.

While the controlling conditions are such as to allow a definite systematic relation between contours, scales, and map distances for the various kinds of surveys, whether for military, geological, city, or subdivision purposes, yet, as indicated by the report of the Committee, there has been absolutely no standardization in these matters accepted by the profession at large.

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† Received by the Secretary, June 3, 1929.

One engineer may make a map on the scale of 1 in. equals 100 ft., for which another one, for exactly the same purpose, may use 1 in. to 200 ft. The one uses 5-ft. contour intervals, and the other uses 10-ft. contour intervals under the same circumstances.

If there were a more general appreciation of the U. S. Geological Survey Standard Topographic Charts, and the amount of information that can be secured from them, much time and expense would be saved, that is now wasted in making preliminary surveys over territory already adequately covered by existing topographic maps.

A trained engineer can secure information from the standard topographic charts which to the uninitiated seems almost unbelievable, and, in fact, impossible. These charts can be and often have been used without modification for such operations as preliminary location of highways, railroads, and even oil-pipe lines, hundreds of miles in length, without the necessity of the engineer even going in to the field for his first studies.

In the same way, detailed plans of extensive battles were planned during the World War on the basis of French topographic maps, with as great accuracy as could have been accomplished by detailed studies on the ground itself.

Another point that cannot be emphasized too strongly in topographic surveys, especially in covering large areas, is the proper and permanent monumenting of triangulation of level and traverse stations with a systematic record on the map sheet showing the correct ground location of these various monuments.

The importance of permanent monuments at frequent intervals cannot be emphasized too strongly in any manual that may be prepared on this subject, and it is something that should be emphasized much more in the technical schools than is being done at present. In the United States probably millions of dollars are wasted every year, due to re-surveys made necessary by inadequate monumenting of previous surveys covering the same area.

It seems to the writer that the Committee has a real field of usefulness in co-ordinating and bringing up to date the best practices in topographical surveying. Much valuable work has already been done in this connection by the U. S. Board on Surveys and Maps, and by the U. S. Geological Survey, the work of which should not be duplicated or overlapped by the work of the Committee in preparing its standard manual on this subject.

HENRY J. SHERMAN,* M. AM. SOC. C. E.—Those who make surveys in a flat country feel like placing emphasis on the importance of accurate leveling in order that surface and sub-surface construction may be done satisfactorily. The speaker believes that the Committees on Control and Construction Surveys should give careful consideration to the necessity for reliable bench-marks. Street grades of 0.1% are rather common in seashore developments so that extraordinary care must be observed in executing them, based on leveling control of a high order.

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The importance of horizontal control surveys also needs stressing. It is almost unbelievable that any one will make a topographic survey of an area already containing horizontal control points, without using them to the fullest extent. In the speaker's practice, control points have frequently been established to limit the errors. Good practice demands this safeguard.

WALLACE H. HALSEY,* ASSOC. M. AM. SOC. C. E.—The speaker feels that in location and construction surveys for highways there is too often a serious lack of study and plan for adequate drainage. On Long Island, with miles of perfectly flat terrain, drainage is a major problem, which, if not attacked at the outset, often causes the pavement to fail due to lack of proper drainage.

State highway engineers as well as those of counties, towns, and other municipalities are equally guilty in this matter of failing to drain the subgrade properly, either through lack of funds to make the necessary studies or through fear of suggesting to the governing bodies the expenditure of relatively large sums for underground structures that cannot be shown to the taxpayer. This is false economy. In many cases in which drainage is not planned and taken care of before construction, it is found necessary to do a makeshift job afterward, which adds to the expense and decreases the efficiency.

In a fast-growing suburban area like Long Island, the natural waterways are being forced into underground structures as the land increases in value. The need is urgent for a comprehensive drainage program covering large areas. In Nassau County, the program should embrace the entire county where now isolated drains are built to take care of local conditions at great expense and with little or no effect on the problem as a whole.

It is the duty of the Engineering Profession to educate the public and public officials to the need of adequate drainage and to the wisdom of providing necessary funds for surveys and studies to that end, as well as for the structures themselves.

J. K. FINCH,† M. AM. SOC. C. E.—The report of the Committee on Topographic Surveys made by Mr. Brown,‡ serves to remind the speaker that the professor in a civil engineering school must occupy what might be called a middle ground in the matter of surveying instruction. The main problem of the teacher is to train men who are going into the civil engineering field in general rather than into surveying in particular. This report has raised criticism of the instruction given civil engineering students in the methods and details of what may be called Federal surveying operations. How far should a civil engineering school go in affording this type of training? With a curriculum already crowded beyond reason it is quite clear that the answer to this question must depend upon how many graduates go into Government survey work; the demand for such graduates; and, to some extent, upon the general educational value of such instruction.

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‡ *Proceedings*, Am. Soc. C. E., April, 1929, Papers and Discussions, p. 912.

It is obvious that the number of engineering graduates entering either the U. S. Geological Survey or the U. S. Coast and Geodetic Survey in a year is less than 1% of the annual civil engineering graduating class of the United States. The work done by these Surveys, although the foremost of its kind in the world, is nevertheless a specialty. The civil engineer in general requires detailed surveys of small areas and, in this work, makes little use of the special methods developed by these Federal agencies for work of a totally different character. Instruction in these methods properly has little place in a general civil engineering curriculum. There is a wide gap in methods and procedure between the civil engineer in general practice who uses surveying to obtain large-scale maps of comparatively small areas, and the Federal engineer whose job it is to map the entire United States. At the same time there are many methods and processes which the Federal engineer has developed that can be used to advantage in "plain" surveying operations; for example, there are triangulation and leveling methods used by the U. S. Coast and Geodetic Survey which are practically essential in modern bridge, tunnel, and city surveys. The civil engineering teacher cannot neglect this type of work; neither should he carry it too far; he must seek the middle ground.

Criticism has also been heard from civil engineers engaged in general practice to the effect that the graduates of engineering schools are not as well instructed in surveying as they should be. The answer is that a very small percentage of the men who are doing the surveying work of general engineering practice are college graduates. Although data have been secured, through a questionnaire sent out by the Committee on Topographic Surveys, which shows that surveying costs frequently constitute the major part of the cost of engineering design, it is true that this surveying work is turned over to men of a lower standard of training than is customary in other branches of engineering practice. The bulk of general surveying in America—to be perfectly frank—is done by men of comparatively poor training and ability.

An interesting example of the effect of this factor in surveying is to be found by comparing the American transit, which is a simple, sturdy, practically "fool-proof" instrument, with the European theodolite, which is a very accurate and delicate instrument of complicated construction. The American instrument was originated for, and made to be used by, comparatively unskilled workers, while in Europe, a Doctor of Philosophy can be secured to operate the theodolite at less pay than the American transitman receives.

Another striking confirmation of this fact was brought out by the questionnaire mentioned* by Mr. Brown. It was found that a large number of engineering offices use the plane-table for making detailed topographic surveys of comparatively small areas and at scales of 1 in. equals 100 ft. or less. This would seem hardly to be justified by the general ideas as to the advantages and use of the plane-table as compared with other topographic methods. The plane-table has been usually described as an ideal instrument for extended topographic surveys by which it is desired to cover a large area at a small scale and with all details omitted except those most important to the particular work. Theory, at least, would indicate the transit and stadia as best for

* *Proceedings*, Am. Soc. C. E., April, 1929, Papers and Discussions, p. 911.

small areas. The answer to this apparent conflict between methods which look best on paper, but are not used in practice, is to be found in the fact that it takes a skilled topographer to do the best class of work with the transit and stadia method, whereas a far less skillful man will produce fair results with the clumsy plane-table. He discovers his omissions and mistakes in measurement and interpretation of contours in the field, and while it may take him longer to do the work with a plane-table, he gets far better results than he could with the more difficult transit and stadia. In short, the best method to use for any survey is not simply a matter of instruments and time. It depends upon the quality of service that can be obtained. The only way to improve the quality of surveying assistants is to offer enough pay to attract the better trained men.

A large part of the criticism of general surveying instruction in the colleges is justified. It is given by the youngest, junior instructors, and frequently it is taught as a purely mathematical subject, sometimes labeled "field geometry". There are mistakes in textbooks on this subject, which never could have occurred if the authors had had actual field experience.

Surveying is exactly similar to all other practical subjects taught in an engineering curriculum in that the economics involved cannot be neglected. An engineer is, or should be, an expert on the investment of funds in engineering enterprises. The surveyor should be trained to secure satisfactory results with the least expenditure of time, effort, and funds. This is expressed by Mr. Brown in the statement:* "In many instances, the engineer is justified in spending only so much of his client's money as may be necessary properly to obtain this special information". To put it bluntly, accuracy costs money. Therefore, it is a waste of effort and funds to attempt to secure a greater degree of accuracy than the purpose of the survey requires. Many surveying instructors will consider this rank heresy, but the sooner they realize that surveying is a practical art and not impure mathematics, the sooner will they begin to turn out men well-trained in surveying.

The speaker cannot help feeling that the report† of the Committee on Control Surveys, excellent as it is, is far above the head or needs of the average surveyor. It is an excellent exposition of modern methods of exact and accurate triangulation, but it includes terms of which the ordinary surveyor is totally ignorant and methods in which he has no interest. No sane person would attempt to tell the personnel of the U. S. Coast and Geodetic Survey how they should do their work. They are experts in this field, and other engineers go to them for advice. The problem of the Committee, in the speaker's opinion, is not to review the work of the Coast Survey, but to improve the quality of general surveying practice. This cannot be done by attempting to broadcast the highly accurate, interesting, and exact methods of the geodetic surveyor.

It is not best to begin at the top and gradually work down from the procedure of primary triangulation through secondary, tertiary, etc., to a system of control which meets the needs of general engineering practice—one that

* *Proceedings*, Am. Soc. C. E., April, 1929, Papers and Discussions, p. 912.
† *Loc. cit.*, p. 923.

would be acceptable to the general surveyor. The Surveying and Mapping Division will have little influence in raising the standards of general surveying practice if it works from the top down. The general surveyor is not prepared to take up the study of geodetic position and the least-square judgment of a triangulation net. On the other hand, he would be interested in knowing something about the development of control for land and topographic surveys which cover only a few square miles. The speaker believes that the Committee on Control Surveys should omit consideration of the problem as applied to National, country-wide surveys—that answer is to be found in the published reports of the Geodetic Survey—and confine its attention to the problem of control as it occurs in general surveying practice. Make it simple and direct, begin with the simpler methods of surveying and "work up".

R. H. RANDALL,* M. AM. SOC. C. E.—One of the principal functions of the Surveying and Mapping Division is to determine proper standards by which to conduct different classes of surveys. In this connection, it occurs to the speaker that this discussion of Committee reports indicates that the need is perhaps not so much for deciding standards or specifications for surveying processes as it is to determine proper standards for surveying results. One of the real difficulties in the field of surveying at present seems to be not that surveyors do not know how to get results, but that they have so many divergent opinions as to just what results are desirable. For instance, in making a location survey, the question of which method to use, whether transit or plane-table, is not fairly answerable until the necessary preceding questions as to purpose and scale of plotting have been considered. After it has been decided that the particular purpose in hand is best served by a map on a scale of 1 in. equals 40 ft., this factor alone would tend to make the map process one in which reliance would be placed mainly on distances taped within a fraction of a foot, and horizontal angles measured carefully by transit. If, on the other hand, the decision is to plot the map on a scale of 1 in. equals 200 ft., such refinements of distance and angle measurement will be lost in the scale of the map. For the latter scale it is obvious that the entirely graphic plane-table method is well adapted. Distances may be measured by stadia, accurate to within 1 ft., and they may be plotted on the plane-table board within the same limit. To attempt to locate points by measurements of greater precision would be to try to secure something that could not possibly be shown on a map of the scale adopted.

A survey, in the engineering sense, is both a means to an end, and, rightly executed, usually an end in itself. Proper performance in all cases must depend quite largely upon a clear understanding of just what is wanted as a final result. The means to be selected to secure this result should be chosen afterward. The speaker believes that the acceptance of the conception of a survey as a thing of permanent value—a capital asset—will increase. When the surveyor has clear ideas as to the final result, and is guided by proper specifications to determine methods by which this result is to be secured, a survey (because of its monuments and its mathematical relationships) may be

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of the greatest possible permanence. The function of the present-day surveyor is not entirely to make repeated, unrelated surveys for individual and specific purposes, but is perhaps primarily to bring into existence a basis of broad surveying information which, when once established, will to a large extent obviate the necessity of many smaller surveys. To such a survey structure he may make additions and revisions; but he will rarely be called upon to re-create it.

The ultimate desideratum is that the area of the entire nation shall be adequately covered by well-monumented and referenced points, the positions of which are known by continental surveys which take into consideration the theoretically correct shape of the spheroid; that based upon these positions every governmental division—certainly every city and town—shall have its own system of surveys and resultant co-ordinate positions; and that every local town or city survey shall result first in the monumenting and giving of accurate plane co-ordinates to every street-line point in the area, and, second, in the co-ordination of every point on every private property boundary. This is the theoretical aim of surveying, as applied to present conditions. Every survey that is executed on a basis which makes it a component part of this ultimate structure is of the greatest permanent value, and constitutes a capital asset in the truest sense.

It is useless to attempt the location of street-line or corner points within closer limits than can be realized by builders erecting structures on such lines. As a matter of practical fact it is not likely that any building, no matter how smooth and enduring its surface structure, can be built more accurately than within $\pm \frac{1}{4}$ in. of the established line. If, then, a survey of any particular area is conducted so that a resurvey will locate the original street-line and corner points with a position error of not greater than $\frac{1}{4}$ in., both the original survey and the retracing survey will fulfill all practical requirements. In suggesting this as a standard of acceptability for this class of survey, it is assumed that in the usual retracing survey some monumented and co-ordinated point may be found within a distance of one block in some direction. If this is not the case, and if for any reason the lines for some large section have been entirely obliterated, then the exact discrepancies between the original position of any point and its subsequent re-establishment will not be of great importance, since the most important requirement is that it shall be in agreement with its surrounding newly located positions.

It would be possible to consider the problem of surveying properly the streets in any section from the other angle, namely, to set up specifications and methods by which a certain precision of position location would probably result, and then comparing this with what presumably is required. As a matter of fact the conclusion as to survey program and methods should ultimately be the same. However, it seems well first to direct attention to the final result to be secured, rather than to proceed in certain ways and then to check the outcome.

It has been suggested that it is well to go slowly in the preparation of reports as to standards of survey results and methods. The speaker believes that this is only partly true. In the preparation of a report on city surveys

by a Committee of the Surveying and Mapping Division, it has been concluded that the need for standards is so great that it would be better to have a report produced in two years which might be considered 90% sound and final in its conclusions, than to devote five or six years to the preparation of one which might be slightly nearer absolute perfection. Progress, of course, may be expected in methods, since these must necessarily change and improve with time. However, the standards of results to be attained are not likely to change materially for some years to come, and it would seem well to discuss these and decide upon them as soon as possible.

There seems to be a trend in city and regional planning that is likely to result in an increase in the amount and quality of surveying work done. Under the Standard Enabling Act for city planning, sponsored by the U. S. Department of Commerce, the speaker believes that the necessity of real, adequate surveys for planning will be increasingly appreciated. It is probable that most of the States will adopt legislation in substantial agreement with this Standard Act. Under such legislation, planning commissions will everywhere be charged with control of the subdivision of land. This control will only be permitted after the commission in each instance has established a master highway plan. At first, such master plans may be so generalized in character as to be nothing more than pictures. Eventually, it will become apparent that in order to prepare such a plan of major streets properly, it is necessary to study and make a well-considered layout of all intermediate or secondary streets. To do this, in turn, as will speedily become obvious, is only possible on the basis of real survey information. The direct subdivision of land within the metropolitan area is so obviously the public's business that it will inevitably and increasingly be done by public agency. In this process an increase in the practice and in the status of surveying is to be expected. More jobs and better ones, for surveyors, should result.

CHARLES B. STANTON,* M. AM. SOC. C. E.—The speaker would like to bring out one thought that has not been mentioned, that is, with reference to the practicing engineer. He does not refer to the surveying expert maintaining a trained staff, but to the engineer who will often let boys do the surveying. That is the problem. What are the colleges doing to educate these boys along surveying lines without necessarily making them engineers? The Carnegie Institute of Technology is equipped to teach such courses, and it has men who attend just to take these specific subjects.

During the Summer Surveying Camp, if possible, the school co-operates with some company, without thought of any financial profit, and arranges to do some practical surveying work, thereby giving the students some very valuable experience under expert supervision. Three minor courses in surveying are now given, but they are concentrated into daily 8-hour periods for two or three weeks preceding the official opening of the school.

One thing that has emphasized the need of surveying instruction in Pennsylvania has been the fight for registration of engineers. Colleges or surveying schools should offer an opportunity whereby men who desire it may obtain

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the information that will make them efficient in surveying, and in that way make them a credit to the profession.

CARROLL F. MERRIAM,* ESQ.—About 1904, the Company with which the speaker is connected made the first comprehensive survey of its project areas. The maps made at that time were lost during the re-organization of the Company, but were found a number of years later, laid away in an old house that had been used for storage. Unfortunately, they were not found until after a second survey had been completed by other engineers. Comparison with subsequent field work has shown that these earlier maps were very accurate, but although the first survey had been carefully done, there was nothing on the finished map except the co-ordinate lines to indicate the quality of the work.

The second survey was controlled for the most part by triangulation but, unfortunately, no permanent marks were left in the field and all the computations and co-ordinates have been lost. Another serious fault is the fact that there are no limits indicating the parts actually surveyed and the parts that have been copied from other maps. A mistake of 100 ft. in the width of the river at a proposed dam site was not detected until several years later, when a more detailed study was made. The third survey, therefore, was necessary and has been carefully triangulated with permanent points set for future reference. It is hoped that this precaution will prove a good investment.

To appreciate fully the advantage of good control for a map, it is necessary to experience the task of assembling one from many different sources. If the exact location of certain known points can be confidently relied upon, the various parts can be pieced together with the assurance that the finished map will contain no serious errors. In the case of a certain dam site for which a general map of the surrounding territory was wanted for plant layout, freight yards, village sites, etc., much field work was avoided by drawing from about ten different sources to supply the necessary details. Furthermore, with permanent triangulation points located in the field, the core boring and other details that are necessary prior to actual construction may be added to the original map at any later date with positive assurance that they are properly located with reference to the previous work.

WARREN C. TAYLOR,† M. AM. SOC. C. E.—During many engineering meetings a discussion arises as to the proper manner in which technical schools should be conducted and as to how some of the subjects should be taught. It was to be expected, therefore, that the engineering school would come in for its share of criticism in regard to surveying.

Most of the suggestions have come from men in practice, who are inclined to look back upon their college training and feel something lacking which they wish had been put in; or their attitude to the fresh young graduate is one in which they expect him to be fully capable to enter their employ as an expert surveyor. In view of this it is well to call attention to some of the problems facing the makers of civil engineering curricula to-day.

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Just as the practising surveyor seeks to have emphasis placed upon his line of work, so other branches of the profession are anxious to have their work recognized. The structural engineer would seek better training in mechanics; or the sanitary engineer, in chemistry and sanitary subjects; or the building constructor, in indeterminate stresses, etc. Every new turn in the profession furnishes a call for a change in the curriculum.

The result is that the courses are overloaded with subjects. For instance, there has been quite a swing of the pendulum in engineering education in the past few years toward business and administration subjects. Some of these are showing themselves in the curricula. This must often be at the sacrifice of some subject already taught. Something must be eliminated or the course must be extended in point of time. The schools which have tried such extension have not found it successful and the tendency to-day is to still keep the four-year course and eliminate subjects if necessary to meet the demands. Surveying has been one of the subjects hard hit by this condition.

The result of the whole situation has been a more thorough study of engineering school curricula than ever before. Every alert school to-day is busy at a revision of its list of subjects taught.

The general trend in answer to the problem has been to simplify the curricula and to place the emphasis upon the purely fundamental matters to the elimination or alteration of the more specialized courses. The plan is to select those subjects which form the basis of almost all branches of civil engineering and to teach these more thoroughly and intensively. This is apparently meeting the approval of many practising engineers as is shown in answer to several questionnaires sent them.

The effect of this movement upon surveying is twofold: First, it has the tendency to reduce the number of surveying subjects taught throughout the four years, particularly the specialized subjects; and, second, it has ranked surveying as an important basic subject.

As to the first, the idea is that instead of having a separate course in all grades and kinds of surveying the courses shall be reduced to two or three strong ones which dwell on the fundamental principles of the subject. Then, let the specialized form come in graduate work or in practise, or, in some cases, as an elective in undergraduate work. From the catalog viewpoint this looks like a reduction in the number of surveying courses and as if the schools were relegating this study to the background. This, however, is not the case, but rather there is a desire to make surveying a truly fundamental subject and to teach students the underlying principles involved rather than to train them to become skillful manipulators of instruments. Rightly taught, surveying can be made not only of keen interest, but co-ordinated with work in advanced engineering subjects, can be made a necessary adjunct to them all.

In regard to the second point, the investigation recently conducted by the Society for the Promotion of Engineering Education as to the subjects which should be included in the civil engineering curriculum resulted in placing surveying in a most prominent place. Both practical men and educators concurred in this; so much so that the Society appointed a special committee to determine the proper co-ordination between school and practise in this study.

In giving weight to the subject it is doubtful, however, if many of these men look upon surveying as possessing its greatest value purely as a textbook subject, but rather they see its broader aspects. Included in this is the discipline afforded a young man as a member of a surveying corps where he must take his position in the party and stand the "kicks" of his fellows. In fact, he must try in some way to get along with the chief of the party. Some of the finest lessons in co-operation come from these experiences.

For accuracy and care there are few experiences which can furnish better training than surveying. As an indicator of the working ability of the young "aristocrat" fresh from college no test can compare with his willingness to take off his coat and get into the hardships and grime of the surveying corps.

These are a few of the elements that are giving surveying its rightful place in American technical schools.

M. W. Weir,* M. Am. Soc. C. E.—Much confusion often results, in surveying, from the lack of a good workable method of compulsory reporting in public records, of outstanding and unmistakable discrepancies of measurements and angles discovered from time to time, in the re-surveys of old lines.

An example is to be found in Jersey City, N. J., where surveyors familiar with the peculiar circumstances recognize three different standards of measure for three different sections of the city. These standards are termed, the long standard, the short standard, and the U. S. Standard. In the case of the U. S. Standard only, are the recorded distances the true distances.

In a mistaken view of good economics, the local surveyors jealously guard the secret of the location of these peculiar sections of the city and, consequently, errors—and their expense—are multiplied on the people.

As a further example, surveys made in Western New York State in 1800, and filed in the public records, remain to-day as recorded more than one hundred years ago, with all the errors of measurement and angle perpetuated, although in many instances these township, range, and lot lines have been re-surveyed several times since. Practically all the old marks of identification of corners have disappeared long since and those whose memories might have carried back to the time when corner identification was possible, also have gone.

Failure to record re-surveys, with corrected distance and angle, has caused and will continue to cause useless expense in the future, if some workable method is not provided to eliminate, at least gradually, from the public records, the perpetuation of misleading and erroneous measurements.

Paul H. Underwood,† M. Am. Soc. C. E.—Of late years, the study of surveying has tended to take a more subordinate position in college courses in civil engineering. Primarily, this may be attributed to changing demands from outside the engineering colleges and to changes in the fields of work of the civil engineer. Years ago the young engineer just out of college was apt to find work in a drafting-room or in a surveying corps. Now (1928) there are more

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fields of employment open. Some graduates go into business, some into sales engineering, and others take positions as timekeepers or other subordinate places in building or other construction work. In many of these positions the young engineer is called upon to do little or no surveying.

As a result of these differences in engineering employment and also because of recent developments in engineering science and practice certain changes in engineering curricula have been inevitable. It has been found necessary from time to time to add new courses and to expand some of the established courses. Since, in the majority of engineering schools, it has not been found practicable to lengthen the engineering course from four to five years, some part of the curriculum must "give way".

As a teacher of surveying the speaker is naturally loathe to see the work in surveying diminished. As a teacher of civil engineering, on the other hand, he recognizes the reasons for it and has even come to look upon it with equanimity. After all, is not the chief purpose of the civil engineering school the training of students in the fundamentals of their profession? Surveying, to be sure, is one of those fundamentals, but the course in engineering has been broadened to such an extent that surveying can no longer occupy as large a place as in former years.

The main duty of the Surveying Department is to train civil engineering students in the methods and economics of surveying, or, in other words, to enable them on graduation to make the more simple surveys, to use maps and surveys intelligently, and to be prepared to judge of the economy of different methods of surveying. There is not time in the regular course to do more than this. To be sure, many institutions provide elective courses to give a more complete training in surveying to those who can elect them. However, the experience at Cornell University has been that comparatively few avail themselves of such opportunities. Until there is further demand from the outside for the training of professional surveyors, it does not look as if the colleges will be able to do much to raise the status of the profession of surveying.

Engineering teachers and practitioners who are especially interested in surveying may well regret this state of affairs. Will they not perform a service to the Engineering Profession if they are on the alert to see that surveying is not crowded into too subordinate a position to give the young engineer adequate college training in this still important branch of engineering?

AMERICAN SOCIETY OF CIVIL ENGINEERS
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PAPERS AND DISCUSSIONS

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PRE-DETERMINING THE EXTENT OF A SEWAGE FIELD IN SEA WATER

Discussion*

By Messrs. George W. Fuller, Kenneth Allen, and R. F. Goudey.

George W. Fuller,† M. Am. Soc. C. E. (by letter).‡—The authors have presented interesting observations with equations corresponding to a variety of assumptions as to the travel of sewage in sea water. Of particular interest is the application of equations to conditions at existing outfalls. It would be advantageous to extend such applications to other outfalls.

From the standpoint of pollution of bathing beaches or shellfish layings it is probable that sewage fields are moved for greater distances at times of heavy wind storms than are generally realized. Public health aspects make it particularly desirable to diffuse the sewage as extensively as practicable and thus eliminate the likelihood of any sewage field at all. Such a result is related to the use of multiple outlets which have substantial submergence, and to the release of sewage under conditions by which its velocity of discharge is quickly and almost completely checked, as well as to its release in numerous small streams whereby quick and thorough dispersion is promoted.

The writer's observations indicate that there is no more adequate design for the discharge of sewage into brackish water than the arrangement provided for the Passaic Valley trunk sewer, designed to serve a population of 1 600 000 and discharging sewage into Upper New York Bay near Robbins Reef Light. The 150 nozzles or diffusers, located about 50 ft. below mean low water and distributed over an area of about 3 acres, release the sewage with a swirling outward motion so that it leaves the periphery of the nozzle in a thin ribbon. Dispersion of the sewage is practically complete.

* This discussion (of the paper by A. M. Rawn and H. K. Palmer, Members, Am. Soc. C. E., published in May, 1929, *Proceedings*, but not presented at any meeting of the Society) is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

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‡ Received by the Secretary, June 6, 1929.

This paper is also of interest to engineers who have to do with the disposal of sewage in the Great Lakes or other large bodies of fresh water where there is opportunity for the discharge to pollute bathing beaches or water-works intakes. There are different factors, of course, in such a situation as compared with outfalls in sea water. Stratification conditions are different and the effect of tides is absent, which in estuaries like New York Bay never allow the water at various depths to come to a complete standstill at the same time. Furthermore, in the Great Lakes, the effect of wind at times may be such as to cause the movement of sewage fields for relatively great distances, thus showing the need of effective dispersion.

KENNETH ALLEN,* M. AM. SOC. C. E. (by letter).†—The phenomenon of the course and diffusion of sewage discharged from submerged outlets is one that has rarely received close study, although the general results have been long recognized. The authors have attacked the problem in such a way that, under given conditions, the form of the issuing jet and its spread over the surface of sea water may be approximately predicted. This is believed to be an important advance over previous investigations along these lines.

The experiments and conclusions drawn therefrom are, however, limited to sea water, and in the application of the deduced formulas it is possible that the inexperienced engineer might not appreciate the important influence of salinity and the direction, form, and diffusion of the sewage stream. In inland cities the phenomena accompanying the discharge of sewage from submerged outlets in lakes and rivers will differ radically from those to be observed in salt water.

The effect of salinity was brought out very clearly in experiments made by the writer for the Metropolitan Sewerage Commission of New York in 1909. Owing to the configuration of the harbor, the flow of upland water brought down by the Hudson River, and the action of the tides, the percentage of sea water was found to average 74.8 at West Bank Light in the Lower Bay and 33.7 in the Hudson River off Fort Washington Point. The extreme readings noted at those points during the year were 93.2% at West Bank Light on November 5, and 0% at Fort Washington Point on April 8. Conditions varied, therefore, from those of fresh water to those of nearly pure sea water.

Several series of experiments were made to learn more of the action of sewage discharged into waters of different specific gravities.

Experiments on Flotation.—Croquet balls were weighted until they balanced in water of a specific gravity of 1.000, and then released in water of known specific gravity at a given depth.‡ The time required for the balls to rise to the surface was noted with a stop-watch. (See Table 5.)

The results, while not entirely consistent, give a fair idea of upward velocities of smooth solids released in waters of different density that will not be

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† Received by the Secretary, June 12, 1929.

‡ The salinity of sea water varies in different parts of the ocean, but for this region chlorine of 18 000 parts per million, corresponding to a specific gravity of 1.025, was assumed as constituting 100 per cent.

reached by a diffusing liquid. Bottles, released in a similar way, rose even more rapidly, on account of their smoother surfaces.

Laboratory Experiments in Diffusion.—A total of thirty-one experiments was made to observe the action of a dyed sewage or water when discharged from a pipette, or small tube, in waters of different salinity. Alcoholic solutions of eosin and fluorescein and aqueous solutions of bluing, permanganate of potash, and uranine were tried. The dye, especially the bluing, was found to increase the specific gravity slightly, and it was feared that alcohol might affect diffusion. Permanganate of potash was likely to be decolorized by organic matter. It was finally decided to use uranine ($C_{20}H_{10}O_5Na_2$) which, in a strong solution, is a deep red, but becomes a brilliant green on dilution, readily detected in a Nessler tube in a 1 : 20 000 000 dilution. A dye called "Special Scarlet" was used later and was found to be a very serviceable and less expensive substitute.

TABLE 5.—Experiments to Determine Upward Velocity of Smooth Solids.

Experiment No.	Number of tests.	Specific gravity of water.	Vertical distance traveled, in feet.	Average velocity, in feet per second.
1	18	1.0215	1.47-1.51	0.500
2	10	1.016	1.70	0.455
3	15	1.007	1.62	0.252
4	15	1.009	1.62	0.301
5	4	1.0195	10.0	0.655
6	10	1.0215	10.0	0.49

These experiments indicated clearly the effect of specific gravity and the way in which a drop or a jet diffused. Some of the forms observed were beautiful as well as interesting: Waves of floc; rings like smoke rings; funnels; and cones and wavy threadlike lines, rising or falling, and diffusing rapidly or slowly. The diffusion of 2 cu. cm. of a 1% solution of uranine added gently to the surface of 200 cu. cm. of waters of varying salinity in beakers and jars began in a few minutes, was complete in an hour with tap water, and took 48 hours in a mixture containing 25% of sea water, while it had not occurred with a 75% mixture of sea water in a week's time. (See Table 6.) Salinity, therefore, was shown to have a marked effect on diffusion.

Experiments in Diffusion in the New York City Aquarium.—These experiments, twelve in number, were made to confirm the results obtained in the laboratory before conducting others on a larger scale in the open water. The tank was 4 ft. high, 4 ft. wide at the top, narrowing to about 3 ft. at the bottom, and 8 ft. long, with an overflow a few inches below the top. The dye and water were fed from a funnel and by a tube to a nozzle, the depth and direction of which were adjustable. Nozzles of $\frac{1}{4}$ in., $\frac{3}{8}$ in., and $\frac{1}{2}$ in. were used.

Without going into the details of these experiments, some of the results were as follows:

(*a*) With a horizontal jet a considerable current may be induced at a slight depth without apparent effect on the surface itself.

(*b*) With a $\frac{1}{4}$-in. horizontal jet of fresh water in fresh water, under a head of 1.0 ft., the axis of the jet remained horizontal, enlarging in

diameter to 3 in. at a distance of 1.25 ft. from the nozzle and to 6 in. at a distance of 3.0 ft. from the nozzle, and then diffusing slowly. After substituting a jet of sewage, the diameter 3.0 ft. from the nozzle was 12 in.

(*c*) In the case of a ¼-in. upward jet of fresh water at a depth of 3.0 ft. in fresh water under a head of 2.5 ft., the jet formed a concave cone 5 in. in diameter 1.0 ft. from the nozzle and spread in a thin layer at the surface. By the end of 5 min. diffusion was quite uniform to a depth of 1.5 ft.

(*d*) With a ½-in horizontal jet of sewage in fresh water near the bottom of the tank, 3.4 ft. below the surface and under a head of 0.15 ft., there was a gradual descent of the jet to the tank bottom which it covered with a blanket about 2 in. thick, with very little diffusion, in 3 min. The temperature of the sewage was 62° Fahr., with a specific gravity of 1.001. The temperature of the water was 61° Fahr., and its specific gravity was 1.000.

(*e*) With a ½-in. jet (nearly horizontal) of sewage in harbor water, at a depth of 3.3 ft. and under a head of 0.15 ft., a temperature of sewage 62° Fahr., specific gravity, 1.0015, and a temperature of water, 58° Fahr., the effluent rose promptly in an upward curving cone, reaching the surface in 1 min., where it spread out. At the end of 3 min. the supply of dye was cut off and by the end of 5 min. there remained but a thin green layer at the surface.

TABLE 6.—DIFFUSION IN WATERS OF DIFFERENT SALINITY.*

Liquid added.		Top water.	Sewage.	Sewage.
Dilution of uranine solution.....		1 : 100	1 : 1 000	1 : 10 000
Specific gravity of liquid added.....		1.000	1.001½	1.001½
Percentage of sea water.	Percentage of tap water.	Diffusion apparently took place between		
100	0	2 and 24 hours.	6 and 48 hours.	.....
75	25	2 and 24 hours.	6 and 48 hours.	Not complete at end of week.
50	50	2 and 24 hours.	6 and 48 hours.	56 hours.
25	75	5 min. to 2 hours.	6 and 24 hours	48 hours.
0	100	At once.	At once.	Few minutes and 1 hour.

* Experiments by Dr. Payne B. Parsons, Bacteriologist, Metropolitan Sewerage Comm. of New York.

(*f*) For a ¼-in. horizontal jet of sewage in harbor water, as in Result (*e*), under a head of 1.0 ft., the diameter in a plane 1.0 ft. above the nozzle was 8 in. (see Fig. 1*). The horizontal component of the jet did not appear to affect the time taken to reach the surface.

(*g*) With a ¼-in. upward jet of sewage in harbor water, at a depth of 2.5 ft., and under a head of 1.0 ft., the sewage rose promptly to the surface with little diffusion and then spread in a thin layer. The diameter 2 ft. above the nozzle was 6 in.

(*h*) At a ¼-in. downward jet of sewage in harbor water, under a head of 1.0 ft., the jet rose as a column of greater diameter and with more rapid diffusion than in Result (*g*) and spread at the surface. Diffusion was increased materially by a slight agitation of the water.

* *Proceedings*, Am. Soc. C. E., May, 1929, Papers and Discussions, p. 1170.

The results of these experiments are generally confirmed by the work of the authors and by a similar apparatus, but their analysis has been carried to a point where the path of a jet of fresh water in sea water may be more closely predicted.

Experiments on Diffusion in New York Harbor.—Nine experiments were made by placing a dye in the surface water of the harbor, or at sewer outlets, to observe the rate at which sewage might be expected to become diffused and dispersed under different conditions. These were followed by twenty-one experiments to observe the result of discharging dyed water at various depths and locations in the harbor.

In three experiments at the outlet proposed at that time for the Bronx Valley Sewer, in the Hudson River, off Mount St. Vincent, dyed water was pumped at rates of 17 to 25 gal. per min., to depths varying from 5 to 25 ft. through a 1¼-in. hose. The velocity of the river water varied from 0 to 3 ft. per sec., and its specific gravity varied from 1.010 to 1.013. The detection of dye at the surface was uncertain at slack water from depths greater than 16 ft., or in a current of 2½ ft. per sec. from depths greater than 9 ft.

Nine similar experiments were made at various points in New York Bay, Gravesend Bay, and the Upper East River. These indicated that:

(*a*) With a difference in specific gravity of 0.004, dyed water will rise in quiet harbor water from depths of 10 to 26 ft. at a mean rate of 0.10 to 0.14 ft. per sec., and be visible at the surface.

(*b*) With a difference in specific gravity of 0.004½, it will rise from a depth of 18 ft., at a mean rate of 0.17 ft. per sec. and be visible at the surface.

(*c*) With a difference in specific gravity of 0.016, it will rise from a depth of 27 ft. in a current of 1½ ft. per sec., at a mean rate of about 0.10 ft. per sec., and be visible at the surface.

(*d*) In moderate currents dyed water may be expected to be visible at the surface if discharged at depths of 15 to 20 ft., but that the rate of ascent is variable. The limit from which it can be discovered under the most favorable conditions appears to be about 40 ft. In sea water (specific gravity, 1.021½), with a current of 0.3 ft. per sec., diffusion is very gradual at first, but proceeds more rapidly later.

Two similar experiments, but on a larger scale, were made in the Upper Bay, near the site of the Passaic Valley Sewer outlet off Robbins Reef, which has since been built.

In the first experiment, 16 500 gallons of a 1:2 750 solution of uranine in tap water was discharged 40 ft. below the surface through a 2½-in. hose, at a rate of 205 gal. per min., giving a velocity of 14 ft. per sec. The specific gravity of the harbor water varied from 1.0115 to 1.0135.

In the second experiment, 56 000 gal. of tap-water containing 290 lb. of uranine, forming a solution with a specific gravity of 1.003, was pumped from a pontoon through an 8-in. pipe, at a rate of about 2 320 gal. per min., or with a velocity of 14.8 ft. per sec., to a depth of approximately 40 ft., the pontoon and discharge pipe rising as the experiment progressed. The specific gravity of the harbor water was 1.015, and the velocity of its current, as observed by submerged floats, averaged: 0.45 ft. per sec. at a depth of 3 ft.;

1.03 ft. per sec. at a depth of 20 ft.; and 2.85 ft. per sec. at a depth of 50 ft.; from which the mean velocity at the point of discharge was taken as 2.2 ft. per sec.

TABLE 7.—Effect of Depth on Percentages of Sea Water and Dissolved Oxygen near Sewer Outlets in Boston Harbor, Massachusetts.

Date, 1911.	Distance from outlet, in feet.	Outlet.	Depth below the surface, in feet.	Percentage of sea water.	Dissolved oxygen percentage of saturation.
August 29.....	10	Deer Island	1	68	53
			7	89	88
August 30.....	90	Deer Island	New surface to 1	80	80
			2	88	95
			3-60	94	100
September 6.....	1 000	Moon Island	New surface to 1	82	73
			1	82	75
			2	86	90
			3	88	95
			5	90	98
			10	92	99
			15	94	100
September 7.....	500	Moon sland	New surface to 1	80	68
			2	86	78
			3	88	88
			5	88	94
			10	90	97
			20	92	100
September 11.....	1 000	Moon Island	New surface to 1	78	68
			2	82	76
			3	86	87
			5	88	92
			10	92	97
			15	94	100
September 11.....	4 600	Moon Island	New surface to 1	80	72
			2	84	85
			3	88	92
			5	90	97
			10-20	94	100
September 14.....	2 000	Moon Island	New surface to 1	82	68
			2	86	73
			3	88	85
			5	90	92
			10	94	97
			15	96	100
September 14.....	2 500	Moon Island	New surface to 1	84	72
			2	86	80
			3	88	86
			5	90	97
			10	94	100
			15	96	100
September 18.....	5 280	Moon Island	New surface to 1	86	75
			2	88	82
			3	90	89
			5	90	97
			10	94	100
September 18.....	1 000	Moon Island	New surface to 1	82	68
			2	86	74
			3	88	85
			5	90	92
			10	92	97
			15	92	98

In spite of these large volumes no evidence of the discharge was observed in either case, although it had been detected once before in one of the smaller experiments at this point and at the same depth. The inference was that diffusion occurred rapidly due to subsurface currents at the time of the larger experiment, and that the rise of sewage to the surface probably depends more on the velocity of such currents than on the volume of sewage discharged.

It is clear that salinity of a body of water receiving sewage is an important factor in its diffusion, as it also is in its oxidation, its effect on precipitation, its decomposition, its general appearance, and its accompanying odors. The sewage of the City of Washington, D. C., discharged in 30 ft. of fresh water in the Potomac River and that of Hamburg, Germany, discharged at a depth of 30 ft. in the Elbe, is difficult to discern at the surface, while that from the Boston and New York outlets forms a field on the surface that is often clearly visible, well defined, and extensive. It was noted in the New York experiments with dyes that the edge of the field on the windward side was most clearly defined, while the more rapid diffusion took place on the leeward side.

From what has been said it is evident that diffusion is promoted by locating inlets in deep water of low salinity, but having a high velocity of flow. Discharge should be horizontal and from multiple outlets. The submerged outlets from Deer Island in Boston Harbor, with fourteen horizontal orifices, and the one hundred and fifty outlets of the Passaic Valley Sewer, in which the upward-flowing stream is dissipated by spiral flanges, are excellent examples of effective diffusion and dilution of an effluent in salt water.

As the visibility of sewage fields in calm weather is largely due to the oily surface film known as "sleek", it is obvious that for the sake of appearance, oily wastes should be kept out of effluents so far as practicable: First, by control and treatment at the source of industrial wastes of this nature; second, by providing municipal collection of crank-case wastes from garages; and, third, by installing skimming tanks on the line of outfall sewers. Other points that might be emphasized are the very great effect of turbulence of current on diffusion, and the variable effect of winds on the shape and extent of the sewage field.

Table 7* shows how the sewage discharged at the Deer Island and Moon Island outlets in Boston Harbor concentrates near the surface, as indicated by the increasing salinity or percentage of sea water and saturation with dissolved oxygen with increasing depth.

R. F. GOUDEY,† ASSOC. M. AM. SOC. C. E. (by letter).‡—The authors are to be congratulated on their presentation of basic data derived from unique and ingenious experiments, which now definitely removes "judgment" as the sole factor to be considered in choosing sizes and lengths of ocean outfall sewers. Although the paper abounds in involved mathematical discussion, which most sanitary engineers shun, an impartial and critical analysis indicates the logic behind the important and significant conclusions which the authors have drawn from their experimental data.

Disposal of sewage and the degree of prior treatment required in connection with ocean disposal need particular study in Southern California, because of general unfavorable conditions, as compared with the Atlantic Coast. Instead of a Gulf Stream, with good currents relatively close to shore, as

* Determinations by Dr. Payne B. Parsons, abstracted from the Report of the Metropolitan Sewerage Comm. of New York, 1912, Chapter III.

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‡ Received by the Secretary, July 17, 1929.

prevails along the Atlantic, California's main current, the Japan Current, runs more than 1 000 miles from shore. The littoral currents are wind-induced, low in velocity, variable in direction, and undependable for sewage disposal. The wind direction, likewise, is variable, due to changing from land to ocean breezes daily. For instance, during the night the land breezes prevail from north to east, and, finally, work around to the southeast in the early morning. With the overcoming of the land breezes by the ocean winds during the forenoon, the prevailing and strongest winds of the day come from the southwest. Toward evening, they veer toward the north, when the land breezes dominate. Except during stormy periods, when the winds are either from the north or southeast, this continual routine of "boxing the compass" accounts for the lack of uniformity in dependable shore currents. For these reasons, the authors are correct in using calm periods as critical.

A rather high standard of ocean disposal has naturally resulted in Southern California, as its coast line alternates with wide sandy beaches, which are ideal for bathing, and rocky bluffs, which are fast being utilized for picturesque home sites. Experiments to correlate bio-chemical oxygen demand, dissolved oxygen, and chlorides, with sewage fields, failed to establish a sensitive relationship or a law. Experience indicates that the sewage field loses its identity as badly discolored water at a *B. Coli* index of 10 per cu. cm., and this is the standard now used by sanitary engineers in California. It is believed to be a reasonably fair basis as compared to the accepted drinking water standard, considering relative quantities of water imbibed for drinking purposes, compared with that imbibed during bathing, nor does the figure differ from the *B. Coli* content common to natural streams having no unfavorable hygienic record. Adoption of less stringent standards, say, 50 *B. Coli* per cu. cm., which might safely be allowed during "slips", is not of much economic importance, because of the closeness of the two zones. Further requirements of ocean disposal dictate that there should be no odor in the vicinity of the outlet, nor sewage litter stranded alongshore.

In the earlier outfalls in Southern California, the length from shore was set arbitrarily, even in the case of large outfall sewers, such as that for Los Angeles. The lengths of the more recent outfall sewers have been gauged from the behavior of *B. Coli* fields of pollution determined from pre-existing outfalls. The outlets are generally made to discharge from up-turned ells lying on the ocean floor, in order to avoid their becoming clogged with shifting sand, and to prevent the transmission of water hammer.

Table 8 gives a few examples of long outfalls in California that have a vertical discharge.

The outfall is rather long, compared with those one would design by use of the formulas given by the authors.

For small communities, the ease of using pure and protected wrought-iron pipe as large as 18 in. in external diameter, has not caused any particular hardship, in achieving the lengths of outfalls actually used. Although the life of such outfalls may not be more than twenty-five years, for growing cities, they represent an easy way to obtain a serviceable outfall for a fairly long term of years.

In larger cities, the cost per foot has been as much as $150, and, in some proposed projects, the cost has been variously estimated up to $500 per ft. Obviously, any factors that may shorten the outfall, or establish its performance in advance, deserve analysis. Investigations to perfect ocean disposal are quite as important as the efforts of inland communities to refine their high-grade treatment processes. Along recreational and residential shores, the ocean should not be regarded as a crude dumping ground for sewage.

TABLE 8.—LONG OUTFALLS WITH VERTICAL DISCHARGE.

City.	Population, present.	Treatment.	Outfall.		
			Diameter, in inches.	Length, in feet.	Depth, in feet.
Los Angeles	1 250 000	Screening	84	5 400	63
Orange County	60 000	Screening	42	2 700	40
Santa Monica*	50 000	Screens	24	2 000	25
Santa Barbara	40 000	Screening	42	3 694	40
Ventura	20 000	Screening	17	2 000	30
Venice	20 000	Screening	14	1 360	25
Santa Cruz	16 000	Screening	2–15	2 000	37
Watsonville	10 000	Screening	16	1 500	20
Oceanside	5 000	Septic tank	11	2 000	33
Mission Beach	5 000	Septic tank	14	1 200	22
Pismo	3 000	Septic tank	11	1 700	17
Pacific Beach	3 000	Septic tank	14	1 500	15
Laguna Beach	2 000	Septic tank	10	600	15

* Downward discharge.

There are certain factors not stressed in this paper which probably have quite a bearing on the performance of ocean disposal outlets. From the time sewage leaves the outlet until it reaches the surface, dilution and diffusion appear to yield an extremely rapid and extensive purification. The greater the oxygen content of the water, and, conversely, the less the oxygen demand of the sewage, the less will be the area of surface pollution. The quantity of dissolved oxygen in sea water varies according to the temperature of the water and the amount of vegetable life in it, which at certain times even causes supersaturation with oxygen of 5 or 10 per cent. The oxygen demand value of the sewage varies with its age and treatment. The behavior of outfalls discharging settled sewage is, in actual practice, much better than those discharging crude or screened sewage.

Added dilution would give lower ratios of dissolved oxygen in the sea water to the oxygen demand of the sewage. Furthermore, the degree of treatment given to the sewage is an important factor in the length of outfall sewer which will be required to keep the sewage-polluted field offshore. Further research into these phases is desirable.

Sludge deposits in the vicinity of outfalls exert high oxygen demand, and, consequently, must cripple seriously the capacity of the ocean to oxidize and dilute the sewage being discharged from the outlet. The apparently large area needed for dilution at the old Long Beach outfall could be easily explained by the high oxygen demand of the sludge, which cut down the dis-

solved oxygen in the sea water, limiting the organic matter that it would handle.

The authors noted a discrepancy in the actual and theoretical diameter of a rising column of sewage.* It is conceivable that this may be occasioned either by the diffusion rate of salt in the sea water as it penetrates the sewage, or the swelling action may be caused by escaping dissolved gases in sewage, which at times might become as much as 20 parts per million expressed as oxygen, and might exert a material buoyant effect distorting the original line of travel. However, the ocean salt would increase dilution, and the sewage gas would shorten the time for initial dilution. There does not seem to be sufficient evidence to warrant a conclusion that the bacteria die in about 5 hours. In fact, there is evidence indicating that the time may vary from less than 1 hour to more than 5 hours. An excellent field of research would be the study of destruction of bacteria in ocean water, including such factors as temperature, oxygenation from dilution, sunlight, and time for various degrees of dilution.

The paper, as presented, has been a great help in extending knowledge on the behavior of ocean outfalls, because it places values on certain factors that heretofore were vague. For instance, in the matter of depth; even for small outfall sewers, Fig. 8† shows that 1 cu. ft. per sec. of sewage, if discharged at a depth of 30 ft., would travel horizontally, from a point above the outlet, 100 ft.; if discharged in 10 ft. of water, the distance of travel would be 2 300 ft. This certainly has a considerable bearing on locating the outfall to obtain the minimum length for the greatest depth possible.

A second point is the desirability of discharging sewage from a horizontal outlet, at a greater velocity than would ordinarily be obtained. The length of outfall can be practically cut in half by obtaining reasonable velocities of discharge. It may be that, by pumping the sewage through a 15 or 20-ft. lift and discharging it through a nozzle, the capitalized cost of pumping would be far less than the cost of an outfall twice its length.

The desirability of multiple outlets is confirmed by these experiments. In the past there has been a debate among engineers as to whether multiple outlets are actually better than single outlets. Table 4‡ seems to afford conclusive proof of the desirability of multiple outlets. With high-velocity discharge, they would be relatively simple to add to many existing outlets that need better dilution and more satisfactory disposal.

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* *Proceedings*, Am. Soc. C. E., May, 1929, Papers and Discussions, p. 1176.
† *Loc. cit.*, p. 1190.
‡ *Loc. cit.*, p. 1191.

AMERICAN SOCIETY OF CIVIL ENGINEERS
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PAPERS AND DISCUSSIONS

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WATER SUPPLY FROM RAINFALL ON VALLEY FLOORS

Discussion*

By Harold C. Troxell, Assoc. M. Am. Soc. C. E.

Harold C. Troxell,† Assoc. M. Am. Soc. C. E. (by letter).‡—Seasonal rainfall cannot be divided into the four groups suggested by Mr. Sonderegger,§ unless another group, that of hold-over storage, is added. During wet years a large percentage of the rainfall might remain in storage to be discharged during the following season.

Just northwest of San Bernardino, Calif., and tributary to Cajon and Lytle Creeks is located Lone Pine Creek, for which records are available since 1919. In contrast to the average Southern California stream, Lone Pine Creek passes over large sand and gravel areas and its canyon side walls are gently sloping and covered with sage-brush. The gauging station is located on an outcropping of bed-rock at the extreme lower end of the canyon. There is every reason to believe that there is practically no underflow by this station. The drainage area above the gauging station is 16.7 sq. miles.

In 1921-22 this area was visited by an unusually heavy rainfall. At a rain gauge located near the month of the canyon 43.23 in. were registered for the season and the run-off from Lone Pine Creek was 3 810 acre-ft. Although this is the surface run-off, it does not represent the run-off from the 43 in. of rainfall. A large proportion of this rain was stored in the canyon and did not drain away for some time afterward.

During the next season of 1922-23 at least 1 440 acre-ft. of the 2 090 acre-ft. run-off for that year was from rain placed in storage during 1921-22. In 1923-24, 530 acre-ft.; in 1924-25, 195 acre-ft.; and in 1925-26, 15 acre-ft. were drawn from rainfall placed in storage during 1921-22. From this it is seen that the seasonal rainfall of 43.23 in. during 1921-22 produced, not 3 810 acre-ft., but at least 5 800 acre-ft. This figure is more likely to be an

* This discussion (of the paper by A. L. Sonderegger, M. Am. Soc. C. E., published in May, 1929, *Proceedings*, but not presented at any meeting of the Society), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

† Asst. Hydr. Engr., U. S. Geological Survey, Los Angeles, Calif.

‡ Received by the Secretary, July 8, 1929.

§ *Proceedings*, Am. Soc. C. E., May, 1929, Papers and Discussions, p. 1140.

under-estimate than an over-estimate, because this storage has had to help supply the transpiration and evaporation losses for at least four years.

In Fig. 6* Mr. Sonderegger has divided the seasons into two groups—those that follow wet years and those that follow dry years. Those that follow wet years are plotted so as to show more run-off than those that follow the dry years.

The accuracy of this graph is questionable, because the entire seasonal rainfall during wet years is plotted against the seasonal run-off for that year, and not against the total run-off due to that rainfall. Likewise, during the following dry years, the run-off is plotted against a seasonal rainfall that only delivered a part of that run-off.

Furthermore, for years following wet years, the transpiration and evaporation losses are shown to be less than for those following dry years. The transpiration and evaporation losses for any season are governed primarily by the rainfall of that season. However, during years following wet years these losses would undoubtedly be greater, instead of less, due to the available storage. Ordinarily, wet years will have greater losses than dry ones irrespective of whether they follow wet or dry years.

For example, Fig. 6 shows that for years of mean rainfall Waterman Canyon Creek will have a transpiration and evaporation loss of 70% following a dry year and only 60% following a wet one. The transpiration and evaporation losses for the year following the wet year should be greater, due to the hold-over storage available to the plant life, and should be less for the year following dry years on account of lack of this storage.

A true division of the seasonal rainfall into transpiration, evaporation, storm run-off, and seepage run-off (penetration) can only be made when the quantity placed in storage during wet years is added to the seepage run-off for that year, and that drained from storage in the following years is subtracted from the seepage run-off for those years. Only one curve will be needed under these conditions. This curve will show the transpiration and evaporation losses irrespective of the character of the preceding years.

Curves, such as Fig. 8,† showing a comparison between seasonal rainfall and storm run-off, should be used with a great deal of caution. Storm run-off depends not on seasonal rainfall, or even storm periods, but on the instantaneous rate of the rainfall and the condition of the canyon storage. Every acre of the canyon has a certain uniform rate by which it absorbs the rainfall if storage is available. If the rate for a given area is 1 in. per hour, then any storm that does not exceed that rate will be completely absorbed. If, for example, 1 in. should fall in ½ hour then ½ in. will run off. If this inch should fall in 15 min., then there would be ¾-in. run-off. Therefore, if a rainfall of 1 in. should occur within the hour, it would be necessary to have the instantaneous rates in order to determine the storm run-off for that rain. On the Santa Ana River the season of 1922 was a good example of this. While the first major storm of that year was as severe as those of April, 1926, and February, 1927, it did not cause any extremely high water.

* *Proceedings*, Am. Soc. C. E., May, 1929, Papers and Discussions, p. 1153.
† *Loc. cit.*, p. 1156.

Another item just as important to storm run-off as the intensity of rainfall is the condition of the canyon storage. If the voids in the canyon walls and gravels of the stream beds are completely surcharged, then the rain will run off as storm run-off irrespective of the rate of rainfall. For these reasons any comparison of seasonal rainfall and storm run-off should be avoided.

Mr. Sonderegger states* that storms in mountain and valley areas of equal size will deliver equal supply, and this is undoubtedly true. Then, by drawing a curve (Fig. 11),* showing the relation of the maximum storms to the seasonal rainfall, he assumes that like maximum storms will produce like seasonal run-off. The logic of this is doubtful because the seasonal run-off (or supply) depends, not so much on the maximum storm, as on the instantaneous rate of rainfall, the distribution of the storms, and the condition of the storage at the beginning of the season.

Table 13 shows the run-off from Waterman Canyon and Strawberry Creeks for the period of record, together with the seasonal rainfall and the storm periods of more than 4 in. at Squirrel Inn.

TABLE 13.—COMPARISON OF RAINFALL AND RUN-OFF RECORDS, WATERMAN CANYON AND STRAWBERRY CREEKS.

Season.	Run-Off.		Rainfall (Squirrel Inn).	
	Waterman Creek, in acre-feet.	Strawberry Creek, in acre-feet.	Seasonal, in inches.	Storm periods of 4 inches, or more, in inches.
1920–21	1 840	3 170	46.15	9.01, 8.40, 7.04
1921–22	6 420	11 000	73.58	23.79, 8.68, 6.91, 6.26, 5.71
1922–23	2 230	3 680	44.27	7.73, 5.01
1923–24	784	1 600	28.42	4.06
1924–25	555	1 430	34.81	4.00
1925–26	1 160	2 520	45.23	19.76, 6.90
1926–27	2 040	3 720	43.32	20.30
1927–28	585	1 300	16.39	5.20

During 1925-26 and 1926-27 the seasonal rainfall at Squirrel Inn was almost the same for both years, as were also the maximum storms. On Strawberry Creek the season of 1926-27 produced a run-off almost 50% greater than that of 1925-26. This was due to several reasons, one being that the latter season was preceded by several dry years and had very little in storage at the beginning of the season. The first storm of the season, a rainfall of 4.97 in. during the early part of October, had practically no effect on the surface run-off. It was apparently used to make up the soil-moisture deficiency and storage. From then until April, the discharge of the creek varied little, except for a short period in February following a storm of 6.90 in. The maximum storm of the season was a 19.76-in. rainfall in April. The storm left the storage in the canyon fairly well filled, insuring a normal run-off for the summer months.

At the opening of the 1926-27 season, while the storage was low, it was hardly as deficient as in 1925-26. Several storms of about 2 in. in November,

* *Proceedings*, Am. Soc. C. E., May, 1929, Papers and Discussions, p. 1157.

followed by storms at intervals of about two weeks, kept the daily discharge about normal. In the middle of February the maximum storm of 20.30 in. occurred. This was immediately followed by several storms of more than 3 in., in March. It was these March storms that caused the run-off for 1926-27 to be greater than the run-off for 1925-26. After the heavy storm of February had filled the available storage in the canyon, the storms of March were forced to run off. Had the April storm of the 1925-26 season been followed by storms such as fell during October and February of that season, the run-off for 1925-26 season might have equalled that of 1926-27.

In 1920-21 the seasonal rainfall was greater than in both 1925-26 and 1926-27 (46.15 in. at Squirrel Inn). The season opened with a storm of 4.00 in. in the latter part of October. From then, until the end of May, followed a series of storms, the greatest of which was a 9.01-in. rain in January, a 8.40-in. rain in March, and one of 7.04 in. in May. All these storms showed a large run-off, due to the fact that the canyon storage was completely charged during most of the rainy season. Unlike both 1925-26 and 1926-27 the 1920-21 season did not have a major storm; yet the run-off from Strawberry Creek was only about 15% less than in 1926-27 and 35% greater than in 1925-26, each of which had maximum storms of about 20 in.

In 1921-22, with a maximum storm of 23.79 in. (only 3.4 in. greater than that of 1926-27), the seasonal run-off was more than three times greater. The heavy storm of December was followed by minor storms at frequent intervals. In other words, it is not so much the size of the maximum storms that effects the seasonal run-off but the order in which the maximum and minor storms occur. Practice in Southern California has taught hydrologists that, while a storm of 3 to 5 in. in October or November might not affect the run-off, storms of 2 to 3 in. following major storms might give considerable run-off.

In order to determine the deep penetration on the valley floor, the only comparison that can be made with the mountain area, is to assume that the evaporation and transpiration losses per unit of area are equal. It seems logical that a natural brush cover in the San Bernardino Valley should have the same transpiration and evaporation losses as the mountain area for like cover.

By drawing curves similar to those of Mr. Sonderegger, showing the transpiration and evaporation losses, and seasonal run-off corrected for storage, for Strawberry, Waterman Canyon and Lone Pine Creeks, the following conclusion might be drawn as to the deep penetration on the valley floor. If the growth was as extensive as that in Lone Pine Canyon where hold-over storage exists, then for a mean seasonal rainfall of 16.15 in. the deep penetration would be about 0.2 in. If the growth were more comparable to that found in Strawberry Creek, then for a mean seasonal rainfall the penetration would be about 0.5 in. On the other hand, there might not be any deep penetration for a season of 16.15 in., if the valley floor was assumed to have a natural brush cover, because, under these conditions, the valley would be in its native state, having large moist areas acquired during wet years. The evaporation and transpiration from such areas would greatly exceed that of the mountain areas.

MEMOIRS OF DECEASED MEMBERS

JOHN JARRETT ALBERTSON, M. Am. Soc. C. E.*

DIED NOVEMBER 23, 1928.

John Jarrett Albertson was born in Magnolia, N. J., on August 16, 1858, the son of Chalkley and Annie (Stokes) Albertson. He was descended from old Quaker pioneer stock and the estate on which he lived has been continuously in the Albertson family since 1687, when it was first settled by William Albertson and his wife, Hannah (Druit) Albertson.

John Jarrett Albertson was graduated from the Friends Central High School, at Philadelphia, Pa., in 1876, and for a year, following his graduation, taught school in his home district.

In 1878, he began his life work as an engineer. He was employed by the Philadelphia and Atlantic City Railroad Company on preliminary surveys and, later, on the construction of this railroad to the Jersey Coast. On the completion of this project, he was associated with Judge John Clement, an eminent jurist, and at that time the leading surveyor of South Jersey.

Mr. Albertson took particular interest in road building and was one of the early authorities in New Jersey on Telford, macadam, and gravel roads, writing many of the original specifications governing these types for various State Highway Commissioners. Prior to 1898, he was Engineer for Camden, Gloucester, and Atlantic Counties, New Jersey, engaged on State Aid road work exclusively. At this time, the law required that the County Engineer reside in the county that he represented. Accordingly, Mr. Albertson gave up his work for Atlantic and Gloucester Counties, retaining only his position as Engineer of Camden County, to which he had been appointed in 1892. He held this position until his death.

In 1893, he went to Europe to study road building. After traveling for a year, he returned with much valuable information which he proceeded to utilize in his own work. The location of many of the principal highways of Camden, Atlantic, and Gloucester Counties, remain as they were originally laid out by him, with very minor subsequent deviations.

Probably his foremost contributions to South Jersey were the Meadow Boulevard at Atlantic City, N. J., in the construction of which he introduced hydraulic dredging for road work; the White Horse Pike which has world-wide fame; and the very fine road system of Camden County. He served as Engineer of the Commission in the construction of the Camden County Court House, one of the finest public buildings of its time, and also was influential in the construction of the Camden County Institutions at Lakeland, N. J., where expenditures in recent years have exceeded $4 000 000. He also served with distinction on the Advisory Board for the New Jersey Approaches to the Delaware River Bridge and as a member of the Technical Advisory Committee of the Philadelphia Regional Planning Federation. In addition, he

* Memoir prepared by Beale M. Schmucker, Assoc. M. Am. Soc. C. E.

acted as Engineer for many municipalities in Camden County, particularly the Boroughs of Hadden Heights and Audubon which he served from their inception until his death.

Mr. Albertson was a Mason of high standing, having been a member of Cyrene Commandery and Crescent Temple. He was also a member of the Camden Lodge of the Benevolent and Protective Order of Elks.

He was a man of varied interests. With his brother, he owned and cultivated the homestead farm. He was President and principal owner of the Defiance Fruit Company and also of the Atlantic Cranberry Company, the latter being one of the largest in the State. His home at Magnolia is a horticultural Paradise and here he raised beautiful and rare plants and trees from all climes. His love of horticulture was carried to the County Institutions at Lakeland where former swamp and waste land has been drained and replaced by beautiful shrubbery and flowered gardens.

Mr. Albertson was also active in the line of finance, having been a Director of the First Camden National Bank and Trust Company which is the largest bank in Southern New Jersey and also a Director in the Woodbury Trust Company.

In 1886, he was married to Elizabeth Swift Wills, of Poughkeepsie, N. Y. They had one child, a daughter, Anna Mary, now Mrs. Lester Collins, of Moorestown, N. J.

Mr. Albertson was a forceful man of high character and great energy, whose absolute integrity was never questioned throughout a long and active public life. He was loved and respected by those with whom he came in contact, and his death was a great loss, not only to the community, but to all South Jersey, as he was one of the foremost citizens of the State.

Mr. Albertson was elected a Member of the American Society of Civil Engineers on March 14, 1916.

ALEXANDER BROCINER, M. Am. Soc. C. E.*

DIED NOVEMBER 16, 1928.

Alexander Brociner was born at Galatz, Roumania, on April 21, 1880, the son of André and Rosalie Brociner. His early life was spent in his native city, where he obtained his elementary and higher education, receiving his Bachelor's degree in 1898. Five years later the Royal Technical College at Bucharest, Roumania, from which he was graduated with highest honors, conferred on him the degree of Civil Engineer. He then served for a time as Second Lieutenant of Heavy Artillery in the Roumanian Army.

While he was still a student, Mr. Brociner had visualized the United States as the country offering the widest scope for his activities. Accordingly, shortly after his graduation, he came to America.

His practical engineering experience began with the firm of Bradford Lee Gilbert, in New York, N. Y., for which he soon became Chief Engineer,

* Memoir prepared by Jacques S. Negru, Esq., New York, N. Y., and Rudolph P. Miller, M. Am. Soc. C. E.

serving in this capacity from 1904 to 1907. At this time Mr. Brociner began his independent practice as Consulting Engineer, on building and heavy construction. He was soon rewarded with success. For more than twenty years, he had an extensive practice, during which he greatly increased the scope of his work, having to his credit apartment houses, hotels, loft buildings, churches, school buildings, railroad stations, water-front structures, bridges, etc. Among the buildings for which he was Consulting Engineer are the Windsor Hotel, Montreal, Que., Canada, the Chateau Laurier and the Terminal Station, Ottawa, Ont., Canada, Lawrence College, Yonkers, N. Y., the King Cotton Hotel, Greensboro, N. C., and the Proctor Memorial Bridge, at Proctor, Vt.

At various times Mr. Brociner contributed technical articles to *Engineering News-Record, Insurance Press, Safety Engineering,* and its predecessor, *Insurance Engineering,* the *Real Estate Magazine,* and also to the New York daily press.

He was an untiring worker who never spared himself when his time and energy were needed. His work was especially marked by a thorough conscientiousness in the interest of his clients. In his particular field of structural engineering, he was a close student of new developments, and his thorough knowledge of good practice and legal requirements made his services of great value. The high regard in which he was held by his profession is expressed in a memorial resolution of the Structural Engineers' Society of New York, of which he was a member, in the following terms, as

"An engineer of ability and highly respected both for his accomplishments and also for his high standard of professional ethics.

"He deserves high credit for his labors in the Society and in an effort to create a better understanding and bond of fellowship among his co-workers."

He is survived by his wife, Jolanthe, to whom he was married in 1909, and by a son, Victor.

Mr. Brociner was elected a Member of the American Society of Civil Engineers on August 28, 1922.

STEPHEN SANS BUNKER, M. Am. Soc. C. E.*

DIED DECEMBER 15, 1928.

Stephen Sans Bunker, the son of David A. and Jeannette (Higgins) Bunker, was born at Bar Harbor, Me., on August 20, 1876. His education was obtained in the Bar Harbor schools and at the University of Maine in Orono. He was graduated from the latter institution in the Class of 1897, receiving the degree of Civil Engineer. He was a Member of Phi Gamma Delta Fraternity.

Immediately on the completion of his college course Mr. Bunker went to Rockland, Me., where for several months he held a position in the City Engineer's Department. He then accepted a position as Transitman on railroad construction with the James T. McDonald Company of New York, N. Y.,

* Memoir prepared by Oliver L. Hall, Esq., Bangor, Me.

remaining with that firm for three years. During this period he was employed in the construction of the Washington County Railroad, now a branch of the Maine Central Lines, and on the Seaboard Air Line in Virginia.

In 1900, he returned to Bar Harbor, where he opened an engineering office and was actively engaged in building an addition to the water-works of the town. From 1901 to 1902 he was employed with the W. G. Wilkins Engineering Company, of Pittsburgh, Pa., holding the position of Resident Engineer in street railway and paving construction. In the fall of 1902 he became connected with the Jersey City, N. J., Water Supply Company in the building of a 6-ft., steel, water main for that city. From 1902 to 1904 he was Chief of Party and Resident Engineer on railroad construction in Oklahoma with the William Kenefich Construction Company.

In 1904 and 1905 Mr. Bunker was in Bolivia, South America, as Chief of Party on preliminary surveys, for the Bolivian Government, in the Andes Mountains. In 1905, he returned to the United States and for three years was employed in the South as Chief of Party and Resident Engineer on railroad work for the Louisville and Nashville, and Chesapeake and Ohio Railroad Companies. In the fall of 1908, he again went to South America, this time to Brazil, where as Division Engineer he assisted in the building of 250 miles of railroad in the Amazon Valley. The road commenced at the head of navigation in the Madeira River, 1 600 miles from the sea, and ended in the wilderness on the banks of the Mamore River, the boundary between Brazil and Bolivia. This work was completed in 1912, at which time Mr. Bunker returned to the United States, going to Toledo, Ohio, where, for several months, he was engaged in street railway surveys.

In the fall of 1913 he accepted a position with the Maine State Highway Commission as Assistant Engineer. He was engaged in this work when the United States entered the World War. He immediately volunteered for service and was sent to the first Plattsburgh Camp in June, 1917. He was not without "peace-time" military experience as he had been Captain of the Color Company at the University of Maine and, later, a member of the National Guard of the State of Maine. Mr. Bunker was soon given a commission as Captain and sent to Fort Sheridan, Illinois, where he was a Drill Master.

He went to France in November, 1917, as Captain of the 503d Engineer Regiment. His first service was in forestry work near the Swiss border, his company being engaged in manufacturing lumber for the trenches. He was then, during a period of several months, in charge of grading and steam shovel work in the Montoir Railroad Yards near St. Nazaire, the principal port of debarkation.

During his service in France Captain Bunker was also in charge of pouring the concrete for several of the immense warehouses constructed at St. Nazaire. Following the signing of the Armistice he was placed in charge of rebuilding the roads in this district, which were severely worn by army trucks.

Captain Bunker spent twenty months in war service in France, returning to the United States in July, 1919, in command of a detachment of the 72d Engineer Regiment. He proved himself an excellent officer, extremely

efficient, and was held in the highest regard by his men. That his services were appreciated by his superiors is attested by the fact that he was recommended for promotion to the rank of Major, but unfortunately promotions were stopped by the Armistice. He later, however, received a commission as Major in the Reserve Officers' Corps.

Following his return from France Captain Bunker resumed his position with the Maine State Highway Department in charge of the construction of several concrete and gravel bituminous roads in various sections of the State. In January, 1923, by unanimous vote of the Board of Public Works, he was elected City Engineer of Bangor, Me., in which capacity he had charge of the street and highway, park, and sewer departments, with power to appoint his own subordinates. In his six years of service to the City in this position, Captain Bunker was given credit for valued services. During this period the City constructed 10 miles of cement concrete roadway, 3 miles of cement concrete sidewalks, and 4½ miles of new sewers. In these years 190 000 cu. yd. of gravel were used by the Municipal Government. This gravel was obtained on very advantageous terms through the purchase of a large gravel bank.

During his service as City Engineer, the Department was motorized from two pieces of apparatus in 1922 to twenty-five pieces of equipment in 1928, and, to-day, it is rated as one of the most competently equipped municipal departments in New England.

Captain Bunker always will be remembered in Maine as a pioneer in keeping adjacent highways free from snow during the winter. The movement spread by the co-operation of neighboring towns with the City of Bangor until the highways in that section were available for motor use all the year around. The effort was so successful that it was copied by other Maine communities until now there are few main highways in the State which are not cleared following heavy snows.

He was ever ready to extend aid and assistance to other City Departments as well as to local engineers and contractors who found the knowledge attained by him of vast help. Always courteous and kindly, he was held in high regard by the employees of the Engineer Department and by the citizens generally.

Captain Bunker died after an illness of several months in Bangor. The funeral services were held on December 17, 1928, at All Souls Congregational Church in that city. The interment was at Bar Harbor. He is survived by his widow, Euna (Sanford) Bunker, a native of Bath, Me., two daughters, Jeannette Mary, of Boston, Mass., and Ruth Elizabeth, a Junior at Earlham College, in Richmond, Ind., and one son, Paul, in the employ of the Maine State Highway Department. He is also survived by his mother, Mrs. Jeannette Bunker, of Bar Harbor, and two sisters, Mrs. Maurice C. Rumsey, of New York City, and Mrs. Oliver L. Hall, of Hampden, Me.

Fitting testimonial to the worth of Captain Bunker as a public official was given by the following public announcement of his death made by the Hon. John Wilson, Mayor of Bangor:

"Stephen S. Bunker, City Engineer of Bangor for seven years, after several months of failing health, died in Bangor, on Saturday, December 15,

1928. Mr. Bunker had served the city with faithfulness, honesty, and ability, and has left as his lasting memorial, a system of concrete roads on the main arteries of the city, of which the city is justly proud, and which establishes clearly his reputation as one of the great road engineers of New England. He was a loyal soldier in the Great War, a good citizen, and a man of high character with an exceptionally lovable disposition. The City of Bangor has met a great loss and will officially and sadly pay proper respect to his memory on the day of his funeral."

Captain Bunker was elected an Associate Member of the American Society of Civil Engineers on October 4, 1910, and a Member on June 2, 1920.

WILLIAM WALLACE HAYDEN, M. Am. Soc. C. E.*

DIED JANUARY 7, 1929.

William Wallace Hayden, the son of James Francis and Catherine (McKittrick) Hayden, was born on August 7, 1861, at Baton Rouge, La.

Mr. Hayden's professional career began at a very early age, as he served as a Rodman in 1877 and 1878 on the New Orleans, Texas and Pacific Railroad (now the Texas and Pacific Railway). In 1879 and 1880, he was an Assistant Engineer on the location and construction of the Southern Pacific Extension from Lafayette to Cheneyville, La., and, in 1880 and 1881, he was engaged as Assistant Engineer on the Louisville and Nashville Railroad, from New Orleans, La., to Montgomery, Ala.

At various periods during 1877, 1879, and 1880, he pursued studies at the Louisiana State University in Baton Rouge. From 1882 to 1885 Mr. Hayden was in the employ of the Yazoo and Mississippi Valley Railroad Company (now a part of the Illinois Central Lines), which was then known by other names. During this period he was Transitman on surveys and Resident Engineer in charge of bridging and track-laying. In 1886, he served as Division Engineer on the Burlington and Missouri River Railroad for a time and subsequently was employed by the Kansas City, Memphis and Birmingham Railroad Company, first on location and then as Resident Engineer.

Mr. Hayden then returned to the Yazoo and Mississippi Valley Railroad and the Illinois Central Railroad Companies with which he remained for many years. He resurveyed the Yazoo and Mississippi Valley Railroad from Memphis, Tenn., to New Orleans; located and built the Bayou Sara Branch and also located many other lines, including that from Centreville to Woodville, Miss., and one from Wilczinski, to Dahomey, Miss. The lines from Jackson to Ethel, La., and from Jackson to McManus, La., and the line from Minter City to Parsons, Miss., were also included in his construction work. He had charge of all construction from the Ohio River southward on the Illinois Central Railroad and from Memphis south on the Yazoo and Mississippi Valley Railroad until about 1905.

* Memoir prepared by J. F. Coleman, M. Am. Soc. C. E.

He was later appointed Chief Engineer and afterward Assistant to the President of the Mobile, Jackson and Kansas City Railroad Company, after which he entered the employ of the Missouri Pacific Railroad Company, locating the Hickman Branch. In 1906 and 1907, he was Locating Engineer and then Principal Assistant Engineer on the New Orleans, Great Northern Railroad. He next located and built the Lakeview Traction Company Line extending out of Memphis, and was subsequently Chief Engineer of the proposed Memphis and Pensacola Railroad. He was afterward employed by the New Orleans and Northeastern Railroad Company on maintenance of way. After a brief period, however, he left that Company to serve as Pilot Engineer, first on the Gulf Coast Lines and, later, on the Missouri, Kansas and Texas Lines.

Mr. Hayden's health could no longer stand the strain of such an active life. In consequence, he forsook the railways which he had served long and faithfully and in 1924 accepted employment with the Louisiana State Highway Commission. He remained in this position until shortly before his death, which was caused by a severe heart attack.

He was a most genial man who made and kept many friends. He had a keen sense of humor and was ever ready for a joke. He was active and energetic, and devoted to the work of railway location in which he was especially proficient. Indoor occupations were irksome to him, and many of his changes of business were made primarily to escape work which kept him indoors.

He was married on December 16, 1891, to Ruth Davenport, of Port Gibson, Miss., who survives him. He also leaves two daughters, Mrs. C. R. Bostick, of Wauchula, Fla., and Mrs. Talmadge Kinnebrew, of Homer, La.

Mr. Hayden was elected a Member of the American Society of Civil Engineers on November 6, 1895.

HOWARD KINGSBURY HOLLAND, M. Am. Soc. C. E.*

DIED OCTOBER 29, 1927.

Howard Kingsbury Holland was born in Edwardsburgh, Mich., on March 24, 1884. He was the second son of Dr. Marion Holland and Harriet (Kingsbury) Holland. His father was a physician and druggist and his paternal grandfather, Dr. Samuel Ward Holland, was one of the pioneer physicians of Eastern Michigan. His maternal grandfather was Asa Kingsbury, a banker and merchant of Western Michigan.

In 1894, Mr. Holland's parents moved to Cassopolis, Mich., and he was graduated from the Cassopolis High School in 1901. The following summer he entered the employ of the Page Wire Fence Company, at Monessen, Pa., in which position he was engaged at first in the drafting-room and, later, as a helper on the open-hearth furnace.

In the fall of 1902, Mr. Holland entered the University of Michigan, at Ann Arbor, Mich., but left in the late fall of 1903 to accept an appointment as

* Memoir prepared by Louis E. Ayres, M. Am. Soc. C. E.

Committee Clerk in the Senate of the State Legislature of Michigan. In the spring of 1904, he accepted a position with the Reo Motor Car Company, of Lansing, Mich., and remained at this work until the opening of college in the fall of 1905. Mr. Holland was graduated from the Civil Engineering Department of the University of Michigan in June, 1908.

Following his graduation, he was appointed Assistant in the Surveying Camp of the University of Michigan under the late Joseph B. Davis, M. Am. Soc. C. E. After the close of the Camp in the fall of 1908, he accepted a position with Gardner S. Williams, M. Am. Soc. C. E., Consulting Engineer of Ann Arbor, and for the following year was Chief of Party on a survey of approximately 12 000 acres of land for power plants and flowage along the Huron River, near Ann Arbor. From November, 1909, to June, 1910, he was a Resident Engineer for the J. G. White and Company, on the construction of a hydraulic power plant at Sterling, Ill.; and from July, 1910, to September, 1917, he was again associated with Professor Williams, as Resident Engineer and Supervisor of Construction, on a number of water-power and water-works plants.

From July, 1917, until his death, Mr. Holland was a member of the firm of Holland, Ackerman, and Holland, Consulting Engineers, of Ann Arbor, specializing in the design and construction of water-power plants. His last work was in connection with two plants on the Menominee River in Michigan.

Mr. Holland had been prominent in the Masonic orders for a number of years. He was affiliated with the Blue Lodge at Cassopolis, Washtenaw Chapter, Royal Arch Masons, at Ann Arbor, and was a member of the Shrine and a Past Commander of Ann Arbor Commandery No. 13, Knights Templar. He was also a member of the First Congregational Church, of Ann Arbor, of which he was a Deacon at the time of his death.

He was married on September 24, 1914, to Alma Marie Schmid, of Manchester, Mich. He is survived by his widow and three children, Howard Kingsbury, Jr., Roberta Louise, and John Marion, and by his brother, Ray Kingsbury Holland, Assoc. M. Am. Soc. C. E., all of Ann Arbor.

Mr. Holland was popular as a student and was a member of a number of campus societies, including Michigamua, Tau Beta Pi, and Owls; among his classmates he was highly respected. As a citizen, his counsel was frequently sought, and he took an active part in public affairs. As an Engineer, his principal services were rendered in the field, and all who knew him recognized his thoroughness in construction supervision and his sound judgment in those important problems connected with foundations of dams, power houses, and similar structures. Although of a quiet and retiring disposition, he was possessed of a strong and manly character. He will be greatly missed by many to whom he was a genuine and substantial friend, and his untimely passing is a regrettable loss both to his community and to his profession.

Mr. Holland was elected a Junior of the American Society of Civil Engineers on June 1, 1909; an Associate Member on October 29, 1912; and a Member on March 9, 1920.

WILLIAM DOUGLAS HUDSON, M. Am. Soc. C. E.*

DIED FEBRUARY 25, 1929.

William Douglas Hudson was born at Rockbridge, Ill., on April 14, 1880. When he was still a very young child, his family moved to St. Louis, Mo., where he received his early education. In 1907 he was graduated from the University of Wisconsin, Madison, Wis., with a degree of Bachelor of Science in Civil Engineering. Preliminary to receiving his degree he had served as Rodman for James Stewart and Company and as Topographic Recorder and Instrumentman for the United States War Department.

From 1907 to 1909, Mr. Hudson was a Detailer and Designer for the Southwestern Bridge Company, Assistant City Engineer at Joplin, Mo., Office Engineer for the Town of Atoka, Okla., and Estimator and Designer for the Corrugated Bar Company. Even at this early period of his career, the trend of his interest seemed to be toward bridge construction, and during the next seven years (1910-17) he served, first, as Chief Draftsman in the Bridge Department, and then as Assistant Bridge Engineer, for the Missouri Pacific Railroad Company.

At the outbreak of the World War, Mr. Hudson attended the Officers' Training Camp at Fort Sheridan, Illinois, and served overseas with the American Expeditionary Forces, as First Lieutenant in the Third Battalion of the 344th Infantry.

Following the war Mr. Hudson became Principal Assistant Engineer with C. E. Smith and Company, St. Louis, participating in numerous important railroad and bridge design projects. These projects included the American Royal Building, at Kansas City, Mo., and the stock yards structures at Kansas City, and Oklahoma City, Okla.; the plans for the conveyor system, North Market Street Dock, St. Louis; the plant of the Barnickel Company, at Webster Groves, Mo.; and numerous track plans and arrangements in East St. Louis, Ill., in connection with the St. Louis Municipal Bridge.

In 1923, he formed an association with Harland Bartholomew and Associates, later organizing and becoming the active head of the William D. Hudson Company, Civil Engineers. In these capacities Mr. Hudson made engineering investigations and plans, with special reference to railroad, bridge, and harbor work in more than thirty cities of the United States and Canada. In general, these studies were part of a comprehensive city plan, involving wide knowledge and experience, as, for instance, the general railroad plan and grade-crossing elimination study at Toledo, Ohio, which was made the basis of a ten-year program between the city and the railroads, and for which the citizens of Toledo voted $4 000 000 as the city's share of the cost in 1925. In other instances, Mr. Hudson prepared complete designs and specifications for particular engineering projects, such as the Franklin Street Viaduct, in Grand Rapids, Mich., a preliminary estimate of damage for the acquisition of more than $40 000 000 worth of property in Los Angeles, Calif.,

* Memoir prepared by Harland Bartholomew, M. Am. Soc. C. E.

as a part of the Major Street Plan; and the relocation and grade separation of the main-line tracks of the Southern Pacific Railroad, in San José, Calif.

Mr. Hudson died in St. Luke's Hospital, St. Louis, from pneumonia, on February 25, 1929, following an operation which he had undergone three weeks earlier. He is survived by his widow, two daughters, Winnifred and Frances, and a son, George D. Hudson.

Mr. Hudson was a member of the American Railway Association, the Society of American Military Engineers, the Engineers' Club of St. Louis, and the American Legion. He was also a member of Fellowship Lodge, No. 345, A. F. and A. M., in Joplin, Mo., and of the Scottish Rite and Moolah Shrine in St. Louis.

Mr. Hudson was elected a Member of the American Society of Civil Engineers on March 16, 1925.

WILLIAM HENRY LUSTER, Jr., M. Am. Soc. C. E.*

DIED DECEMBER 12, 1928.

William Henry Luster, Jr., the son of William H. and Elizabeth H. (Yates) Luster, was born at Elizabeth, N. J., on October 11, 1863. He was descended on his father's side from old Colonial stock.

Mr. Luster's early school days were spent in Elizabeth, and his later education obtained at Rutgers College, New Brunswick, N. J., from which he was graduated in 1884 with the degree of Bachelor of Science. In 1890, he received the degrees of Master of Science and Civil Engineer from his Alma Mater.

Following his graduation from college, Mr. Luster was engaged from December, 1884, to December, 1887, as Topographer with the United States Geological Survey and from December, 1887, until 1890, as Assistant to the Division Engineer of the Morris and Essex Division of the Delaware, Lackawanna, and Western Railroad Company, in charge of the maintenance of way and construction of 10 miles of new road. In January, 1890, he entered private practice and was engaged on the construction of county roads in Union County, New Jersey, and in the vicinity of New York City. He had practically full charge of four miles of construction work and laid out and graded Colonial Park, N. J., for Mr. J. R. McPherson. During this period, he was also engaged on sewer construction and had charge of the building of coal pockets for Mr. L. M. Palmer, of Brooklyn, N. Y., as well as foundations of the power house for the Edison Electric Illuminating Company, at Bay Ridge, N. Y.

In June, 1896, he was appointed a Topographer with the New York State Land Survey, which position he retained until June, 1899, when he was elected City Engineer for Elizabeth, N. J. In this capacity he had complete charge of the design and construction of public works the value of which was more than $500 000. From January, 1913, to November, 1926, he was engaged in private practice as Engineer for the Towns of Roselle Park, Roselle, and

* Memoir prepared from information on file at the Headquarters of the Society.

Hillside, N. J. He made the plans for the sanitary sewer for the latter town, which finally was completed successfully by him in November, 1928.

Mr. Luster was a member of the Chi Phi Fraternity, the American Road Builders' Association, and the American Society of Municipal Improvements. He was also a Director of the Citizens' Building and Loan Association of Elizabeth and a senior partner of the firm of Luster and Luster. Aside from these affiliations, Mr. Luster was interested in hunting, photography, traveling and fishing, and, as a hobby, took great delight in the making of fine fishing rods. He was a Democrat in politics, a Rotarian, and a member of the Presbyterian Church.

In December, 1889, Mr. Luster was married to Bertha Overton Looker, of Elizabeth, who, with two sons, Clifton H. and Eric W., survives him.

Mr. Luster was elected an Associate Member of the American Society of Civil Engineers on October 3, 1900, and a Member on April 2, 1902.

DAVID EDWARD McCOMB, M. Am. Soc. C. E.*

DIED JANUARY 27, 1929.

David Edward McComb, the son of James E. and Martha (Jorden) McComb, was born on August 23, 1850, at Chelsea, Mass. On one side of the family he was of Scotch ancestry, his father having been born in Scotland; on his mother's side he was of French lineage. His father, a school teacher, died in 1852 and, in 1855, the family moved to Washington, D. C., which thereafter was Mr. McComb's home.

He attended private and public preparatory schools in Washington until he was fifteen years of age. In 1865, he entered the Navy Yard at Washington, as a Minor in Steam Engineering, an educational course provided by the Navy Department at that time for the training of candidates for commissions as Steam Engineers. After three years of this class work, however, Congress provided, in 1868, that future appointments to such commissions should be made only from graduates of the Naval Academy at Annapolis, Md.

Mr. McComb's training, thus interrupted in 1868, had consisted largely in experimental work under William Shock, Chief Engineer, U. S. Navy, on strains in Bessemer steel, and under S. M. Green, U. S. Navy, on calorific conductibility of boiler-plate materials. He also completed a two years' course in higher mathematics and modern languages at Columbia College, now George Washington University, in Washington. From 1868 to 1870, he was engaged under John A. Coli, Civil Engineer, in Washington, on general city work; in 1870, as Assistant to John A. Partridge, on the construction of the Baltimore and Potomac Railroad; in 1871, as Assistant to the late Thomas Doane, M. Am. Soc. C. E., on the construction of the Burlington and Missouri River Railroad, in Nebraska; in 1872, as Topographer on the Jersey Shore, Pine Creek, and Buffalo Railroad, and as Engineer in Charge of the Fredericksburg, Orange and Charlottesville Railroad; and in 1873-74 as Resident Engineer on the Washington City and Point Lookout Railroad.

* Memoir prepared by C. B. Hunt, M. Am. Soc. C. E.

He entered the municipal engineering service of the District of Columbia on September 1, 1874, and continued therein except for a four-year interval, until his retirement on reaching the age limit on August 20, 1928—a half century of professional service to this municipality. The first thirty-four years of this service, from 1874 to 1908, was in the Sewer Division, of which he became Superintendent in 1888. As such, he devised a formula, based on the experimental work of Burkli-Ziegler, modified to suit local conditions, which is still applied as well as the Kutter formula, to drainage design in the Department.

Following extensive inspections of the existing sewage disposal installations in the United States, England, and France, Mr. McComb recommended a disposal project for Washington in 1890, the engineering elements of which were followed, with but little material variation, by the report made later in that year by a board of sanitary engineers appointed from the country at large by the President to consider this subject. The trunk drainage and sewer mains, subaqueous crossing of the Anacostia River, the subaqueous outlet system and sewage pumping station constituting this disposal project were located, designed, and constructed under his supervision.

On May 15, 1908, Mr. McComb was appointed Chief Engineer of Sewer and Paving Work in Havana, Cuba, where he served as such for four years, the appointment having been made during the period of the second American intervention. During his incumbency in this position he practically completed the subaqueous sewer crossing of Havana Harbor, the sewage pumping station and the tunnel under Cabanas Ridge, as well as many of the urban sewers and also the beginning of the paving of the city street system. His selection for this duty was made by Colonel William M. Black, Corps of Engineers, U. S. A., M. Am. Soc. C. E. (now Brigadier-General, U. S. A. (*Retired*)), Adviser of the Department of Public Works of Cuba, based on his knowledge of Mr. McComb's high professional ability and uncompromising integrity of character.

On July 1, 1912, Mr. McComb returned to the municipal service of the District of Columbia as an expert on asphalt paving work in connection with the contemplated establishment of a municipal asphalt plant; at the conclusion of this duty, on January 1, 1913, he was appointed Engineer of Bridges, a position that he held with distinction for fifteen years until his retirement on account of age. Under his supervision during this period were the structural features of the Dumbarton Bridge, the Meigs Bridge, the Benning Road Viaduct, the South Dakota Avenue and Van Buren Street Bridges over the Baltimore and Ohio Railroad, and many wharves, trestles, and analogous works constructed by the municipality.

Mr. McComb's professional work was marked by sound knowledge of principles, conservative design, conscientious care in execution, and remarkable integrity of administration.

His outstanding personal characteristic was rugged honesty. The circle of his friends was large and he held them throughout his life. He was fond of many sports; playing baseball with notable skill; bicycling throughout Europe and many parts of the United States; and in his later years becoming an

enthusiastic golfer. He was affiliated with the Washington Society of Engineers, of which he was a Charter Member, the Capital Bicycle Club, and the Columbia Golf Club.

Mr. McComb was married on August 28, 1877, to Theresa M. Maher who pre-deceased him on March 9, 1924. A daughter, Mary E. (McComb) Fries, survives. He was a devout member of the Roman Catholic Church.

Mr. McComb was elected a member of the American Society of Civil Engineers on February 7, 1877. His period of membership, extending within a few days of fifty-two years in length, was exceptional and notable; and he left but very few living Society members in date of membership senior to him.

GEORGE COTNER MASON, M. Am. Soc. C. E.*

DIED MARCH 10, 1929.

George Cotner Mason, the son of James R. and Mary W. (Cotner) Mason, was born in New York, N. Y., on May 4, 1871. He received his early education in the public schools of New York and was a student in Civil Engineering at New York University from which he received the degree of Bachelor of Science in 1892, Civil Engineer, in 1893, and Master of Science, in 1894.

From 1892 to 1904, Mr. Mason was engaged as Assistant Instructor and Assistant Professor in Civil Engineering at New York University. During this period he laid out its grounds, buildings, and sewer system, and was also employed as Engineer in charge of the erection of the Young Men's Christian Association Naval Branch Building, in Brooklyn, N. Y., the Army Branch Building, at Fortress Monroe, Va., and a residence, stable, and riding ring for Mr. Frank J. Gould, in New York.

In 1904, Mr. Mason moved to Portland, Ore., where he organized the Hurley-Mason Company, serving as its Vice-President and Chief Engineer. The Company was engaged in engineering and contracting work, with offices in Portland and in Seattle, Tacoma, and Spokane, Wash. It completed many important contracts in the Pacific Northwest, notable among which are the Oak Grove Hydro-Electric Plant of the Portland Electric Power Company and Camp Lewis, a 45 000-man Army Cantonment, near Tacoma. The latter was a World War activity and was completed in record time.

The Hurley-Mason Company also constructed many important buildings and structures throughout the Northwest. Among these are the Todd Shipbuilding Plant and the Northern Pacific Railway Station, in Tacoma; the U. S. Veterans' Hospital, at American Lake; the Seattle Construction and Dry Dock Plant, in Seattle; the Sherwood Building and Elks Temple, in Spokane; the Lewiston Hotel, in Lewiston, Idaho; and the Electric, Board of Trade, Gas Company, and Lipman-Wolfe Buildings and the Portland Flouring Mills, in Portland.

In 1926, Mr. Mason retired from the Company and, until his final illness, was engaged in the private practice of his profession, with offices in Portland.

* Memoir prepared by Ben S. Morrow, Assoc. M. Am. Soc. C. E.

He took a keen interest in civic matters and served as Civil Service Commissioner for the City of Portland from 1916 to 1925. He was also active in the affairs of the Society and was particularly interested in the development of the Student Chapters and in the establishment of closer relations between the profession and the students during their college years. He represented District No. 12 as a Director of the Society from 1923 to 1925.

Mr. Mason was an engineer of independent and courageous judgment, thorough and painstaking in all his work, and these characteristics are evident in the many structures built by his Company.

In 1900, he was married to the daughter of a prominent pioneer family of Portland, M'liss McCracken, who, with one son, Henry M. Mason, Jun. Am. Soc. C. E., survives him.

Mr. Mason was elected a Junior of the American Society of Civil Engineers on March 6, 1894; an Associate Member on May 3, 1899; and a Member on September 1, 1908.

FRED FORREST MOORE, M. Am. Soc. C. E.*

DIED JUNE 2, 1928.

Fred Forrest Moore was born in Waltham, Mass., on October 8, 1867, the son of Edwin F. Moore and Mary E. (Small) Moore. He received a part of his education in the public schools, leaving High School in order to prepare himself for the Massachusetts Institute of Technology, from which he was graduated in 1891 with the degree of Bachelor of Science in Civil Engineering. While he was attending college, his work attracted the attention of President Amasa D. Walker, who invited him to join the Teaching Staff, but Mr. Moore preferred the practice of his profession.

Previous to entering college, he had had considerable experience in surveying, and in 1890 and 1891, having been granted a leave of absence during the Institute term, he was in charge of a plane-table party on topographical surveys of lighthouse reservations along the Maine Coast. Directly upon graduating from the Institute, he joined the Engineering Force of the Boston Water-Works, for which he was engaged on the design of the Sudbury Dam and Reservoir. In 1896, this work was taken over by the Massachusetts Metropolitan Water Board, and Mr. Moore continued in charge of designs and estimates of the Headquarters Field Office. On the completion of this work, he became Principal Assistant to the Superintendent of the Sudbury Department with maintenance and operation duties.

From October, 1900, to April, 1902, as Assistant Engineer on filtration experiments for Springfield, Mass., Mr. Moore constructed plant and carried on experiments on slow sand filtration under the joint direction of the Massachusetts State Board of Health and the Consulting Engineer. From April, 1902, to February, 1903, he was engaged on designs for a concrete railway bridge and other structures connected with the Wachusett Reservoir built by the Massachusetts Metropolitan Water and Sewerage Board.

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* Memoir prepared by Franklin H. Robbins, Assoc. M. Am. Soc. C. E.

From February, 1903, to January, 1904, he served as Principal Assistant Engineer on studies, designs, and estimates for the Aqueduct Department of the Burr-Hering-Freeman Commission on Additional Water Supply for the City of New York. During the next three months, Mr. Moore held a similar position with the Northern New Jersey Flood Commission. From April, 1904, to September, 1905, he was Office Engineer, and from the latter date to April, 1906, Hydraulic Engineeer, for the Committee of Twenty of the National Board of Fire Underwriters.

From April, 1906, until his retirement on March 2, 1928, Mr. Moore was a Designing Engineer for the Board of Water Supply of New York City. In this position he was in charge, under the Department Engineer, of studies and designs for cut-and-cover aqueducts, dams, and reservoirs, including landscape work, steel pipe siphons, distribution lines, and sewage disposal works.

He was a member of the Municipal Engineers of The City of New York, the New England Water Works Association, the Massachusetts Institute of Technology Alumni Association, the Masonic Fraternity, the Royal Arcanum, the Arkwright Club, and the Nannahagan Golf Club.

Mr. Moore was admired by his many friends for his kindly disposition, his fair dealing, and for the ability and energy which he devoted to his professional work. On October 5, 1896, he was married to Winnefred T. Wright, who survives him.

Mr. Moore was elected a Member of the American Society of Civil Engineers on April 3, 1907.

L. FREDERICK PATSTONE, M. Am. Soc. C. E.*

DIED JULY 13, 1927.

L. Frederick Patstone was born on June 24, 1876, at Providence, R. I. He attended the Providence Manual Training High School, and from 1895 to 1899 was a student in the office of the City Engineer of Providence. During this period he also studied at the Rhode Island School of Design and took a special course in Civil Engineering under John E. Hill, M. Am. Soc. C. E., at Brown University. He later received the degree of Civil Engineer from Ohio Northern University.

From June, 1899, to February, 1903, Mr. Patstone served as Assistant Engineer in the City Engineer's Office of Providence. Having taken and successfully passed a Civil Service examination, he received a Government appointment as Civil Engineer in the Philippine Islands, which service he began on March 3, 1903. Thereafter, for the remainder of his life with the exception of a brief absence during the World War, he was identified with engineering activities in the Islands and in China.

The early part of this work was in the Department of Engineering and Public Works at Manila. Successively, except for the two years, 1914 to 1916, when he was in private practice as Civil Engineering Contractor, and July, 1918, to July, 1919, when he was an officer in the World War, he was Super-

* Memoir prepared by A. Gideon, M. Am. Soc. C. E.

intendent of Streets and Bridges, First Assistant to the City Engineer, Acting City Engineer, and City Engineer of Manila. He also served as Superintendent of Water Supply and Sewers, and Acting Mayor of Manila.

To recount all the accomplishments under Mr. Patstone's direction is unnecessary; a mere general review of the different kinds of work included will give an idea of the extent and detail involved. He had charge of laying thousands of square yards of cement sidewalk and pavement. Various bridges were constructed under his supervision, including reinforced concrete arches, several highway bridges of more than 200 ft. span, and a steel lift bridge. A large stone-crushing plant, barges for handling the crushed stone, reinforced concrete school buildings, municipal markets, asphalt and macadam roads, a 3 000-ft. sea-wall—these brief items indicate the wide variety of his work.

Early in his stay in Manila he evolved a city plan covering an area of 68 sq. miles. It was his pleasure to see this plan in part developed under his supervision. Many miles of road were thus built. Much water-front land was reclaimed for developing the harbor. The extensive drainage, construction, and sanitary improvements were incidental to this development. Finally, during the latter years of his régime, he was privileged to perfect many of the plans previously evolved, for the future development of this important city.

During the World War, Mr. Patstone, already a Major in the Officers' Reserve Corps of the Army of the United States, was detailed as Colonel and Chief of Engineers of the Philippine National Guard, but he resigned from this position as soon as permission could be obtained, to enter the Engineer Officers' Training Camp at Camp A. A. Humphreys, Va., after which he served as Major of the 217th Engineers, U. S. Army.

In February, 1919, he returned to Manila to resume the office of City Engineer. This position, however, he resigned the following year in order to engage in a commercial interprise in Shanghai, China. He was Vice-President of the Asia Development Company, with headquarters in Shanghai, and personally directed a notable engineering work carried on by the Company in Shantung Province in 1922 and 1923 when a vast area of land was reclaimed and saved from the flood waters of the Yellow River by extensive diking. He afterward became a member of the firm of Wooten and Patstone, Architects, in Shanghai, and was thus engaged when he was taken with his last sickness. He died at the Shanghai General Hospital on July 13, 1927, after an illness of three months.

Mr. Patstone was a member of the following technical societies and clubs: The Boston Society of Civil Engineers; the Elks, and the Army and Navy Clubs, of Manila; the French, American, Union, and Columbia Country Clubs, and the Chinese-American Association of Engineers, all of Shanghai.

In addition, he was also a member of the American Chamber of Commerce of Shanghai, for which he served as Chairman on the Committees of Architects and Engineers. He was a Member of the Benevolent and Protective Order of Elks and a Thirty-second Degree Mason, Member of Afifi Temple, A. O. N. M. S., Member of Bamboo Oasis of Manila, and President of the Nomad Oasis in Shanghai. He served as a Member of the Traffic Commission of Shanghai, and, after that body was dissolved, he became a Land

Commissioner, having been a nominee of the Shanghai Municipal Council for that position. He also served with the Shanghai Volunteer Corps, as a member of the American Mounted Troop.

As Mr. Patstone devoted the best years of his professional life to the Manila service, it is there that he was best known and best appreciated for his true worth. His success was due not only to his technical and administrative ability, but in no small measure to his sympathetic appreciation of the real mission of the American Government employee in the Philippines: "to train the man under you to fill your position". Certainly he did that; he trained a Filipino as his own successor as City Engineer of Manila. Not until, himself at the top, he saw the City Service practically Filipinized, did he feel free to seek the greater financial gain which private practice had to offer.

In addition to his technical ability, Mr. Patstone was an accomplished musician and was endowed with a remarkable singing voice. He was always ready to use this talent for charity concerts and the pleasure of his friends.

He was married on March 21, 1908, at Calapan, Mindoro, Philippine Islands, to Daisy Suter Clark, the daughter of Captain J. J. Clark, U. S. Army, who survives him.

Mr. Patstone was elected a Member of the American Society of Civil Engineers on April 2, 1913.

YSIDORO YGNACIO POLLEDO y SANDS, M. Am. Soc. C. E.*

DIED MAY 7, 1928.

Ysidoro Ygnacio Polledo y Sands was born in Matanzas, Cuba, on July 31, 1865. A student in his native city, as well as in Germany and France, he also attended the School of Mines at Columbia University, in New York City, and was graduated with the degree of Mining Engineer in 1885.

Following his graduation, Mr. Polledo served as Assistant Engineer on the survey for a water-works for the City of Santiago de Cuba, Cuba. In 1886, he was appointed Assistant Engineer, and, later, Principal Assistant Engineer, in charge of track and structures for the Cárdenas and Júcara Railroad Company, Cárdenas, Cuba. From 1889 until 1890 he acted as Manager of the Santa Bárbara Plantation and Sugar Factory, at Baró, Cuba, later becoming General Manager for the Cárdenas Sugar Refining Company, at Cárdenas.

In 1895, Mr. Polledo again varied his field of occupation, when he was appointed Assistant Engineer on the Harbor Works of Havana, Cuba, which position he held until the following year, when he engaged in private practice as a Consulting Engineer. In this capacity, he re-designed sugar houses, and the evaporating, condensing, and water-cooling plants for sugar factories in Santa Bárbara, Elizalde Santa Catalina, Alava, La Luisa, Carmen de Crespo, and Narcisa, Cuba.

In 1899 he again became Assistant Engineer on the Harbor Works at Havana, under the United States Military Government, Division of Cuba, Department of Havana. During subsequent years, until 1920, Mr. Polledo

* Memoir prepared by Dra. Cira Polledo, Matanzas, Cuba.

was employed in the following positions: Technical Director of the Santa Lucia Sugar Factory, Gibara, Cuba; General Manager of the Matanzas Railroad Company; Manager of the "Banco Español de la Isla de Cuba"; Consulting Engineer in private practice and Vice-President of the Ferro-carril del Norte de Cuba. From 1920 until his death, he served as Vice-President of the Regla Coal and Oil Company.

Mr. Polledo was a man of strict integrity which was applied in his profession as well as in his public life. He is survived by two daughters.

He was a member of the Cuban Society of Engineers, the American Institute of Mining and Metallurgical Engineers, and the American Society of Mechanical Engineers.

Mr. Polledo was elected a Junior of the American Society of Civil Engineers on January 2, 1889, and a Member on November 4, 1903.

HENRY GOSLEE PROUT, M. Am. Soc. C. E.*

DIED JANUARY 26, 1927.

Henry Goslee Prout was born in Falls Village, Va., on August 10, 1845, the son of William and Amanda (Goslee) Prout, and a direct descendant of Timothy Prout who came from Biddeford, Devonshire, England, and landed in Boston, Mass., in 1644.

As the family had moved from their Southern home to New York and, later, to Massachusetts, Mr. Prout's boyhood was spent entirely in Northern surroundings. His education, prior to the Civil War, was not a very systematic one, but was gleaned intermittently at Stockbridge (Mass.) Academy and under the private tutelage of the Rev. Franklin Le Barron. In 1863, at the age of 18, he enlisted in Company D of the 57th Regiment of Massachusetts Volunteers, and served with the Army of the Potomac in the Wilderness Campaign, the siege of Petersburg, and the resulting pursuit of General Lee. In 1865, he was mustered out of service and after a two-year period of adventure and study, entered the University of Michigan, at Ann Arbor, Mich. He was graduated in 1871 with the degree of Civil Engineer, later receiving the honorary degrees of Master of Arts from Yale University in 1902 and Doctor of Laws from the University of Michigan in 1911.

During his second year in college Mr. Prout commanded an expedition of reconnaissance in Southwestern Colorado and so distinguished himself by his astronomical achievements that he was brought to the notice of Gen. William T. Sherman, then at the head of the U. S. Army and previously in command of the Military Division of the Missouri. In consequence, Mr. Prout was one of six young men who, at the request of the Khedive of Egypt, was recommended for military service by General Sherman. He was commissioned as Major of Engineers and remained in Egypt four and one-half years, during which time his experiences were both humorous and tragic, and his strength and endurance were given the most severe of physical and mental tests. In these years of service he rose to the position of Colonel in the General Staff and

* Memoir prepared from information on file at the Headquarters of the Society.

became Governor General of the Provinces of the Equator, relieving Gen. Charles George Gordon when he went to Khartoum as Governor General of the Sudan. An interesting problem which Colonel Prout undertook while he was in the Equator Provinces was the transportation of an 80-ft., screw-propelled steamer which was to be erected at Dufli, across 60 miles of broken country on the backs of negro bearers. It was in this steamer that Emin Pasha was sent to meet Stanley at the time of the famous "rescue".

On his return to America, Colonel Prout was employed as Signal Engineer by the Toucey and Buchanan Interlocking Switch Company, of Harrisburg, Pa., which later developed into the Union Switch and Signal Company. He remained in this position, however, for only a little more than a year, and in 1880 entered the printing business with the firm of Atkin and Prout, publishers of *The Railroad Gazette*. In March, 1887, he assumed the Editorship of this journal, which position he held with distinction for sixteen years.

At the expiration of this time, his interests were again directed to the Union Switch and Signal Company with which he had been associated and, in consequence, in 1903 he resigned his position as both Editor and Director of *The Railroad Gazette* to accept the office of First Vice-President and General Manager of the Union Switch and Signal Company, at Swissville, Pa. He resigned this position in July, 1914, shortly after the death of the late George Westinghouse, M. Am. Soc. C. E., in whose general business interests he had been an adviser, and in November of that same year became President of the Hall Switch and Signal Company, of Garwood, N. J.

In 1915, Colonel Prout retired, giving up at this time all active business pursuits. It was during this period of his life, while he was living at his home in Summit, N. J., that he wrote the well-known biography of George Westinghouse, published in 1924, which, in addition to being an accurate portrayal of its subject, also sheds great light on the character and personality of its author.

In his diversified career, Colonel Prout was an outstanding figure in military, editorial, and professional achievement. His early associations with the great world figure, General Gordon, his startling editorial read on the floor of the United States Senate to which was due the choice of Panama instead of Nicaragua as the route for the Isthmian Canal, and his skill in mercantile leadership and its allied associations with the switch and signal companies—all lend unending tribute to his name. A distinguished scholar and a gentleman, he is well described in the following sketch of appreciation by his life-long friend and associate, Mr. William H. Boardman:

"The sum of it is that he is inspiring. He has such a fund of general and specific information, such a love of truth and thoroughness, such a scorn for lying and carelessness, such a ready quality of human sympathy that, with a faculty for memory cultivated and quickened by editorial habit, he can subordinate himself and skillfully direct conversation so as to draw out others."

Colonel Prout was a member of the American Geographical Society, the Sigma Phi Fraternity, the Century Club, the Railroad Club, and the University Club, of New York, N. Y.

He was married at Fort Leavenworth, Kans., in 1877, to Gabriella Perin. He is survived by his widow, two sons, Henry B. and Curtis, and three daughters, Elizabeth, Mrs. Pierpont V. Davis, and Mrs. Paul G. Tomlinson.

Colonel Prout was elected an Associate of the American Society of Civil Engineers on November 6, 1872, and a Member on September 3, 1879. He served as a Director of the Society from 1893 to 1895.

JOHN RICHARD RABLIN, M. Am. Soc. C. E.*

DIED SEPTEMBER 17, 1928.

John Richard Rablin was born in Plymouth, England, on October 25, 1868, the son of William H. and Mary Edwards Rablin. He came to the United States with his parents when he was about three years of age.

He was graduated from the Boston Latin School, Boston, Mass., and entered Harvard College, but the death of his mother, which caused the removal of the family to Maine, ended his college education.

Mr. Rablin began his engineering work in 1888 as an Engineering Assistant on the Old Colony Railroad, under the late B. T. Wheeler, M. Am. Soc. C. E. His professional education was obtained under Mr. Wheeler's tutelage, and by studying nights by himself, while he was working during the day. When Mr. Wheeler went to the City of Boston as Superintendent of Streets in 1895, Mr. Rablin went with him, remaining there until some time in 1896, when he became associated with the contracting firm of Horton and Hemenway, as Contractor's Engineer. This firm had contracts in connection with the construction of the South Terminal and Back Bay Stations. The principal work on which Mr. Rablin was engaged was the construction of the Back Bay Station.

His work with the Metropolitan Park Commission commenced in 1899, when he was made Assistant Engineer in charge of construction of the Charles River Speedway. In 1904, he was appointed Chief Engineer of the Commission, a position which he retained until his death.

There was a tremendous expansion in the amount and kind of work done in the Metropolitan Park System after Mr. Rablin assumed the position of Chief Engineer. The work was not only construction and maintenance of parks, but also the construction of many important boulevards and their later improvement to enable them to carry heavy automobile traffic, as well as the construction of a number of bridges in the district. The great diversity of his work would be better understood, perhaps, if it were mentioned that he also had charge of shore protection and the construction and maintenance of sea-walls at various points along the shore in the general vicinity of Boston.

Among the works which Mr. Rablin constructed were the Revere Beach Parkway; a dam at Newton Lower Falls, Mass.; the Lynn Shore sea-wall; the Nantasket Beach sea-wall; shore protection at Revere and Winthrop,

* Memoir prepared by Lewis E. Moore, M. Am. Soc. C. E.

Mass.; dam, tide-gates, and boat lock at Medford, Mass.; the Northern Artery; and approximately twenty-eight bridges, the majority of which were spans of considerable size. Among these are the new bridge on the Old Colony Parkway, the Larz Anderson Bridge, over the Charles River, and the Cottage Farm Bridge.

He was continually reading and seeking knowledge regarding up-to-date methods and the latest developments in engineering along all the various lines into which the wide diversity of his work led him. He showed a great deal of ingenuity in varying types of construction to avoid duplication.

The following appreciation by Chairman Keniston, of the Metropolitan District Commission, was printed in *The Boston Transcript* shortly after Mr. Rablin's death:

"An able and devoted servant of the Commonwealth, John R. Rablin, passed away this week, Mr. Rablin became Assistant Engineer to the Metropolitan District Commission in 1899, was appointed Chief Engineer in 1904, and continued in that capacity until his death. During that time the construction of many public improvements in metropolitan parkways, bridges, and reservations came under his direct care. The wise planning and satisfactory completion of these public works was largely due to his engineering skill and attention.

"Mr Rablin's life work was the development and maintenance of the Metropolitan Parks. He was ever ready to give freely of his time and strength, and his early death may be attributed in part to his attempt to resume too soon his duties after a serious illness. He still maintained an interest and oversight of his work as long as his strength permitted. The name of the engineer may not be known or remembered by those who enjoy the benefits of the public improvements for which he was responsible, but the works themselves will remain as a tribute to his earnest and successful efforts. May the public occasionally think gratefully of such men as Mr. Rablin."

Mr. Rablin was a man of very decided character, strong in his likes and dislikes, and much beloved by his friends. He was a member of the Society of American Military Engineers; the Boston Society of Civil Engineers; the American Road Builders' Association; the Massachusetts Highway Association; the International Road Congress; the Boston City Club; and the Wollaston Golf Club. He attained the ultimate in golf by "making a hole in one."

On November 15, 1893, Mr. Rablin was married to M. Frances Loring. He leaves, in addition to his widow, a son, Richard Loring Rablin.

Mr. Rablin was elected a Member of the American Society of Civil Engineers on August 31, 1909.

JOHN CHESTER RALSTON, M. Am. Soc. C. E.*

DIED JULY 15, 1928.

John Chester Ralston was born in Kincardine, Ont., Canada, on May 12, 1864, his parents being James Glendenning Ralston, of Paisley, Scotland, and Mary Amanda (Johnston) Ralston, of Otterville, Ont., Canada.

* Memoir prepared by J. A. L. Waddell, M. Am. Soc. C. E.

His father was descended from an old Scottish family, the McDuffs, Thanes of Fife of Renfrewshire, the earliest records thereof being dated 1190. The last son of that ancient family, Ralph, on receiving a grant of land, named a part of it for himself, Ralph's Town, afterward abbreviated into Ralston—hence, the family surname.

Both Mr. Ralston's father and grandfather were graduates of the University of Edinburgh. His mother was a descendant of the Johnston and Harris families of Massachusetts and Rhode Island, noted in the history of the Minute Men of the Revolution. One of her earlier forebears is credited with being the author of the first book ever written in America, a quaint volume entitled "God's Dealings with Zion's People".

Mr. Ralston received his early education in one of the public schools of Ontario, under a Head Master who afterward became Minister of Education for the Dominion of Canada. There, the foundation was laid for a liberal academic course to be developed in either a Canadian or an American university; but these plans were suddenly terminated by the death of his father, who left him, at the age of thirteen, the eldest of nine children. Thereupon the inspiration of the classroom had to be replaced by the harsher curriculum, of self-support. Having seen the metes and bounds of his father's property re-surveyed, young Ralston developed a nebulous hope of becoming a surveyor and ultimately an engineer.

In 1880, he went into the West and found early employment in the United States Engineer Corps, under the direct supervision of Mr. Charles S. Pease, on the reclamation of the Missouri River, at the beginning of the Federal improvements on the Missouri, Mississippi, Ohio, and their tributaries. It was under the encouragement and personal help of Mr. Pease and his Staff that Mr. Ralston's engineering self-education was directed and inspired. The members of the Staff were largely graduates of the Rensselaer Polytechnic Institute, who thus exemplified in their lives a generous spirit of service.

Under this tutelage the curriculum of the Rensselaer Polytechnic Institute was followed by difficult, arduous, and, at times, most discouraging effort. The nature of the pioneering and improvement plans of the Missouri River, however, was ever fascinating and inspiring, and the discouragement of lamplight study was always transmuted during the day into Aladdin's mysteries. The wizardry of the laws of hydraulics, the nebulous latitudes of river regions and channel erosions, the numberless problems, and the never-ending devices for control or remedy, were a challenge to engineering study. Under such special environment the college classroom as an educating medium was surpassed.

In later life Mr. Ralston undertook a broadening, non-technical course of study, following the curricula of Columbia University and the University of Chicago.

In 1882 he was transferred from the Missouri River at Omaha, Nebr., to the Potomac River, on the reclamation of the Potomac Flats at Washington, D. C., and Georgetown, Md., harbors, and on the building of what afterward became Potomac Park, Washington, D. C. He was given responsible charge of the construction of the mattress foundation of the sea-wall behind which the channel dredgings were deposited to form the terra firma of this interest-

ing park. This sea-wall was more than five miles long, and required several seasons to complete. The wall defined the final north shore line of the main channel of the Potomac River from Georgetown Harbor, past the historic Long Bridge, to a confluence with the south shore of the Washington Channel entrance to the city docks, enclosing a peninsular area that separates the two main navigable channels. This area reclaimed a large and offensive part of the mud flats of the Potomac River, while the dredgings from the widened outside channels supplied the necessary material to raise the flats well above high tide and to create a fertile acreage for landscape gardeners. More than 400 acres were thus reclaimed.

When, in 1885, the major part of the reclamation had been fairly completed, Mr. Ralston was tendered a position as Assistant Engineer on the Construction Staff of the Union Pacific Railroad. For the first two years of his railroad experience he was employed directly by the late Jacob Blickensderfer, M. Am. Soc. C. E., Chief Engineer, principally in collaborating the history of the Pacific Railroads, on the building of which Mr. Blickensderfer, together with the late Grenville M. Dodge, Hon. M. Am. Soc. C. E., and Adna Anderson, M. Am. Soc. C. E., had served so conspicuously.

Minor executive duties also were required of young Ralston in the line of reports, and in furnishing supplies for the many field parties of location and construction in the several Western and Northwestern States crossed by this road. Most valuable experiences were gained and studies pursued under the kindly and inspiring direction of this new Chief, who always took the liveliest personal interest in the welfare and education of his assistants. Three nights a week were devoted to study and instruction in the astronomical observatory of Mr. Blickensderfer, whose work in this field was well and favorably recognized by the United States Astronomical Observatory at Washington. Here the principles of railroad engineering and the early basis of railroad economics, in both engineering and operation, were studied with a dogged determination and with a lively appreciation, in the realization that a master of this temper and vision was not to be found every day.

In 1887, a new Chief Engineer was appointed in the person of the late Virgil G. Bogue, M. Am. Soc. C. E., who early granted Mr. Ralston his long cherished wish to be put into the field. An extended experience in preliminary and final location in many of the rugged Western States, including Colorado, Wyoming, Nevada, Western Montana, Eastern Washington, and Idaho, and on construction work in Kansas, Nebraska, Colorado, Wyoming, and Montana, gave a diversity and comprehension of the widest application of railroad engineering, such as seldom come to young men. Mr. Ralston was given charge of the remodeling of the Butte City, Mont., yards to meet the growth of the expanding mining industries of that region. In that work he was called on to examine and report on the first water-power sites contiguous to the smelter region that later was to be so remarkably developed.

Other notable work arising out of that time and region was the preliminary and final location of the high-line development of what, in later years, became the famous Butte, Anaconda, and Pacific Railway, the first electrically-

operated, heavy-freight carrying railroad in the world. To these works and allied activities Mr. Ralston, as Assistant Engineer, contributed with distinction.

In 1889 and 1890, he took charge of the preliminary surveys and much of the final location of the west end of the Pacific Short Line through Wyoming and Utah, employing and directing a half dozen field parties. He succeeded in shortening the main-line distance 90 miles between the Missouri River and Ogden, Utah, and in finding a most economical crossing of the Continental Divide, nearly 1 200 ft. lower than that of the near-by competitor. Across the uplands of the Big Sandy drainage he was able to locate a long tangent on a 0.5% grade, at exceptionally low cost, the east end of which was within eight miles of the Continental Divide. In 1890 the first period of his career ended, when a suspension of active railroad development in the West was declared.

Mr. Ralston immediately took up steel skeleton building construction, foundation work, and elevated railway erection in Chicago, Ill., and pursued this class of occupation continuously until 1897. During this period he erected a number of early steel skeleton buildings and special structures, especially for the Illinois Steel Company in South Chicago, and for the Lake Street Elevated Road, including the down-town loop along Lake Street to and down Wabash Avenue to Harrison Street, etc. This erection constituted the first down-town loop, and was quickly followed by the joint loop structures, located in what has since developed into the most congested metropolitan district of the world.

Several notable steel skeleton structures were erected in Buffalo, N. Y., Pittsburgh, and Philadelphia, Pa., including the Ellicot Square Building, in Buffalo, and the Presbyterian Building, in Philadelphia, each notable in its day for size and construction.

In the midst of these activities, in the spring of 1897, Mr. Ralston was called to Spokane, Wash., by some of his earlier friends of Butte, Mont., to assist in certain mineral developments and in the construction of reduction works. He assisted in the design and construction of the largest plant of its kind in the State of Washington; and in this, the third period of his career, he established himself as a Consulting Engineer, his practice quickly growing into substantial proportions. It covered examinations, reports, and valuations in the fields of hydraulic development, hydraulic power, and later, hydroelectric power in British Columbia, Washington, Oregon, Idaho, California, Montana, and Nevada, including a fascinating diversity of technology.

Intense activities in those fields, especially in California, led to an examination of the water supply and power possibilities of the Toulomne and its Hetch Hetchy Valley, the Stanislaus, and the Mokelumne, out of part of which, and certain parts of the Merced, was finally carved the so-called Hetch Hetchy water supply for San Francisco, Calif. Another interesting work was his study of the hydraulics involved in the disposal of the underground waters of the famous Comstock Lode, their siphonic behavior, vadose waters, accumulations, and colonization. A year was spent on this work and on the plans for the complete unwatering of certain of the leaner parts of the Lode,

involving a blending of the technology of the Civil Engineer and the rudiments of Economic Geology.

In April, 1906, the San Francisco earthquake destroyed much of that beautiful city and caused Mr. Ralston to be sent to the hospital, where a major operation became imperative, and rendered him a permanent invalid. This put an end to field activities for all time, and left to him as a residue only a limited consulting practice. Notwithstanding these almost insuperable physical handicaps, he was able to take up the duty of re-organizing the Engineering Staff of the City of Spokane, Wash., which was tendered him at that time. He succeeded in bringing into that sadly depleted organization an *esprit de corps* and a technological staff of singular merit, one comparable to the best metropolitan standards. Under the atrophying influences of indurated city politics, a brief but decisive encounter followed, resulting in the establishment of a permanent and high-minded Engineering Corps. Under this régime Mr. Ralston was able to replace eight old and dilapidated wooden Howe truss bridges by designing and building an equal number of reinforced concrete structures, literally making Spokane the "City of Bridges". Among these structures is the famous Monroe Street Bridge, said by many to be the most monumental concrete arch bridge in the United States. The main deck is a segmental arch of 290 ft. span, with a rise of 140 ft. Several of the other bridges are notable in design, and are representative of the true arch bridge.

Large paving areas were inaugurated for the first time for the city; and a new underground water supply of exceptional quantity and purity was designed and built under Mr. Ralston's direction; thus the struggling Western outpost was transformed into a modern metropolitan city. This city-building activity demanded a liberal and wide technology, with a generous spirit of service on the part of the entire staff.

Mr. Ralston was ever an earnest advocate of the principle that the engineer always should take the leadership in professional, civic, and political life. He maintained that the engineer by native mental qualities and education is more fitted for leadership in the activities of men and society than the lawyer, the divine, or the physician. He especially insisted that the engineer should have a command of forceful, fluent, and happy diction commensurate with the leadership to which he should aspire. He lived to see the day when the best thought in the profession is becoming vocalized and insistent on this subject, in direct contrast to two or three decades ago when the engineer was seldom an advocate on any subject except engineering.

In respect to the use of the English language, Mr. Ralston practiced what he preached; for both his speech and his writings were conspicuous for an extensive vocabulary and a fluent diction. Occasionally, some of his brother engineers smiled over his use of long or unusual words, but those words were always well chosen, apropos, and forceful.

During his invalid years from 1906 until his death, aside from his work as City Engineer, he found enough physical ability to report on a number of hydro-electric projects and on straight hydraulic developments aggregating 750 000 h.p., in units varying from 3 000 to 50 000. Prior to the entrance of the United States into and during the World War, Mr. Ralston served as

a member of the Naval Consulting Board for the State of Washington—a period extending from 1915 to 1918. During this time he was also one of about 2 000 entrants who were asked to submit plans and specifications for the dissemination of American propaganda among the German people. His plans were accepted; and he was one of the first four to receive Presidential recognition and a monetary reward.

He also served as a member of the Committee on Development of the Society, sharing actively in its deliberations during 1918-1920. From 1920 to 1924, Mr. Ralston served as a member of Engineering Council. In this Council he represented, as the Executive Member, the States of California, Nevada, Oregon, Idaho, Washington, and Montana.

From 1900 until 1918, he acted as Consulting Engineer, and, for a part of that time, as Vice-President, of the Pacific Coast Pipe Company with a factory at Seattle. This Company manufactured all sizes of wood-stave pipe, notably the continuous-stave type, up to diameters of 120 in. Its use was mostly for penstocks and pressure pipe for hydro-electric plants and for inverted siphons in irrigation. He was one of several engineers of the Pacific Coast who developed and made popular this pipe for more extended use, and this work properly may be said to have been a great contribution to the current engineering development of the time.

He was an early worker in the Boy Scout movement, a pioneer in social hygiene work, a deep lover of the arts, and ever willing to aid in the betterment and beautifying of his city, having planned outlines for the economic and beauteous development of the falls and river of Spokane. He was President of the Art Association, and a brilliant, original, and much sought speaker. He never feared to tell the truth, possessing a decided vision of life and of engineering, which also found expression in his great enjoyment in writing. He was interested in and always ready to assist young men.

While Mr. Ralston was certainly a self-made man, he differed essentially from most of the American engineers of this class. Not only was he a great reader and student, but his studies were laid out systematically and were followed religiously, so that eventually he was in no whit inferior to the leading alumni of the best technical schools.

For two decades the writer had held Mr. Ralston in the highest esteem, having greatly admired him as a man, a citizen, and an engineer—as, truth to tell, did every one else with whom he was thrown into intimate contact. In corroboration of this statement, quotations from a few of the letters of sympathy received by Mrs. Ralston will serve.

That from an associate reads:

"He was a fine citizen, a capable and resourceful engineer; and as a friend I have not known his superior."

That from a subordinate is:

"The great pleasure of working for and with 'Ralston' was the being in contact with a man of vision; for to him engineering was not the mere application of formula and instrument, but combined with these the foresight to conceive projects which should lead men's feet over the hills to an era."

Of the Humboldt County, California, Project, it was stated:

"We undertook a very daring project; and, thanks to his skill, accomplished it with brilliant success. The North Mountain Power Project was one of the first of its kind in the West, and its record for time has never been beaten."

M. E. Cooley, M. Am. Soc. C. E., wrote:

"My acquaintance with Mr. Ralston dates from his service on the Board of Federated Engineering Societies. I learned then of his sterling qualities, his high ideals, and the inspiration of his presence."

That from L. W. Wallace, President, Federated Engineering Societies, read:

"He lived a life of rich accomplishment and endearment, a rich heritage to his friends. He will be mourned and missed in the Engineering Profession, to which he contributed a great deal."

A College President stated:

"His understanding of his associates was an indication of his very generous and universal kindliness."

A Bishop of the Protestant Episcopal Church wrote:

"He was a rare man. He was a true and loyal man in all the relations of life. His ability, courtesy, and consideration for others always impressed me greatly."

Mr. Ralston's day is past; his work is done; but it will be many, many years before he is forgotten. The history of such a life should be an inspiration to young men who desire to become engineers, but who are unable to secure money for a regular course of technical instruction.

Mr. Ralston was married in New York, N. Y., in April, 1897, to Mary Kean Buckner, of Kentucky, whose family came from England to Jamestown, Va., in Colonial days. He is survived by his widow, a son, John W. B., a daughter, Mary Elizabeth, two brothers, and five sisters.

Mr. Ralston was a member of the Spokane Engineering Society, the Seattle Engineering Society, the American Institute of Mining and Metallurgical Engineers, the Cosmos Club of Washington, D. C., the Spokane City Club, and the Sons of the American Revolution. He was also a Thirty-second Degree Mason, a member of the Protestant Episcopal Church, and a staunch Republican.

Mr. Ralston was elected a Member of the American Society of Civil Engineers on October 3, 1906.

BEVERLEY STROTHER RANDOLPH, M. Am. Soc. C. E.*

DIED FEBRUARY 5, 1929.

Beverley Strother Randolph was born on July 17, 1851, at Berkeley Springs, Va. (now West Virginia). He was the eldest son of the late James Lingan Randolph, M. Am. Soc. C. E., Chief Engineer, and, later, Consulting

* Memoir prepared by O. Robbins Randolph, Esq., Rutherford, N. J.

Engineer, for the Baltimore and Ohio Railroad Company, and the late Emily (Strother) Randolph, a sister of Gen. David Hunter Strother.

Mr. Randolph attended Cornell University for two years and then went into active civil engineering work. He was engaged for some time on location for the Baltimore and Ohio Southwestern Railroad Company (now a part of the Baltimore and Ohio System). He then went to Nicaragua on the construction of the first railroad to cross that country. On his return to the United States, he was appointed Superintendent of the mines of the Consolidation Coal Company, with headquarters at Frostburg, Md., which position he held for twenty years. During this time it is interesting to note that not a single strike occurred among his men.

In 1904, Mr. Randolph retired from active engineering work to his farm at Berkeley Springs, W. Va., where he bred horses for the New York market. He later moved to "Tonoloway" near Hancock, Md., where he continued raising horses and also operated a glass sand deposit on his farm. In 1927, at the age of 76, he sold "Tonoloway" and bought a farm, "Hawkhurst", in Berkeley County, West Virginia, near which he was killed in a grade crossing accident on February 5, 1929.

Mr. Randolph was a man of remarkable physique and at the age of 72 was still schooling horses to jump. At the time of his death, he was making plans to go into the breeding and training of trotting horses on a more extensive scale than he ever had attempted before.

He was a Member of the Order of the Cincinnati, and a Life Member of the American Institute of Mining and Metallurgical Engineers.

On September 20, 1882, he was married to Mary Jewette, of Baltimore, Md., who died on December 26, 1918. He is survived by his three nephews, James Robbins Randolph, O. Robbins Randolph, and Lingan Strother Randolph, Jr., and one niece, Mrs. Emily Randolph Deitrick. His brother, the late Lingan Strother Randolph, M. Am. Soc. C. E., died in 1922.

Mr. Randolph was elected a Member of the American Society of Civil Engineers on May 2, 1888.

THEODORE CHARLES ROBERTS, M. Am. Soc. C. E.*

DIED SEPTEMBER 28, 1928.

Theodore Charles Roberts was born in New Orleans, La., on December 3, 1875. He spent his boyhood there and, simultaneously with his early school days, worked in a system of shops owned by his father, thus acquiring a knowledge of mechanics and manufacturing, together with his elementary education. Following his public school preparation, his education was continued at Tulane University, at New Orleans, and at Colorado College, Colorado Springs, Colo. He was a tireless student both through his school and college days and throughout his life. Were it a problem in mathematics or mechanics, a question of industrial or political economy, the design of an

* Memoir prepared by W. R. Palmer, M. Am. Soc. C. E., and John G. L. Cunningham, Esq., New York, N. Y.

industrial plant, the location of a railroad, the management of a high enterprise, the purchase of equipment, or a game of bridge, his ever active mind was alert and eager.

His dissipation, like that of many other engineers and authors, was in his work. His rise in the industrial world was rapid. Always eager for mental advancement, he spent approximately six years teaching mathematics while serving as Field Engineer and Construction Superintendent for the Canon City, Light, Heat and Power Company, as Manager for several combined exchanges for the Colorado Bell Telephone Company, and as Construction Superintendent and Engineer for the Telluride Power Company, Telluride, Colo. At this time, while he was associated with Mr. Ralph D. Mershon, a number of high-tension electrical principles were established.

Subsequent to these engagements, Mr. Roberts became Manager of several mining and milling companies in Colorado. With the experience accumulated up to this time he entered the field of Consulting Engineering and Contracting, at Leadville, Colo. His keen insight, strong personality, and executive ability created a demand for his services with the United States Reduction and Refining Company as Engineer in charge of all design, construction, and operation for its various plants.

From 1904 Mr. Roberts served two years as Chief Electrical Engineer for the American Smelting and Refining Company, in charge of all electrical equipment. His efficiency here caused his transfer to the position of Chief Engineer and Master Mechanic in charge of engineering and mechanical operations for the Pueblo, Colo., Plant.

In 1908 the electrical industry had exceeded all expectations and left the works inadequate for the demand. This exigency drafted Mr. Roberts into the position of Chief Engineer and General Superintendent for the Arkansas Valley Railway, Light, and Power Company, at Pueblo, Colo., a property of H. M. Byllesby Company, of Chicago, in charge of the rebuilding, expanding, and bringing up to date its steam and hydro-electric power plants, street railways, high and low-tension transmission lines, and the electrification of various industries to be served by them.

By 1912 the expanding copper industry in Arizona had outgrown entirely its reduction plants which caused a complete re-organization of operations. The United Verde Copper Company, one of the properties of the late Senator Clark, as did the others, decided on this expansion and engaged Mr. Roberts for the work. It consisted of a new reduction plant, for 9 000 000 lb. of copper per month; a housing development for 10 000 people, with the attendant roads, streets, public utilities, steam and electric railroads, and land reclamation; as well as the management of the operations for the existing plant. In this position his wide experience and ability brought him into all the large engineering developments of the State.

With the beginning of hostilities in 1917, and prompted by his patriotic spirit, Mr. Roberts offered his services to the United States Government. He was commissioned a Major in the United States Army and became engaged in the manufacture of dyestuffs and chemicals.

Soon after the Armistice was signed he became associated with the phonograph industry, as Assistant to the President in Charge of Engineering and Manufacturing for the Columbia Graphophone Manufacturing Company, where, under his direction, the production of the plant was increased from $15 000 000 to $100 000 000 per year, gross sales. Unfortunately, the heavy responsibility and concentrated application for many years had so broken his health that thereafter he had to limit his services as a consultant to a few selected clients. His liberal disposition and interest in others caused him to devote himself through his spare moments to scientific and other writings in order to perpetuate as far as possible his broad experience.

Mr. Roberts died as simply as he had lived. He attended his club in the afternoon, passed the evening with his family, and joined the staff of his Creator very quietly before dawn on September 28, 1928.

In the world of recreation Mr. Roberts' intensive work was broken now and then by a skillful game of golf, a friendly bout with the gloves, a wild-game hunting trip, a cruise in his yacht, or a clever game of bridge. He was a member of the American Society of Mechanical Engineers, the American Institute of Mining and Metallurgical Engineers, the American Institute of Electrical Engineers, the National Electric Light Association, the American Institute of Architects, the Engineers and Columbia Yacht Clubs of New York, the Country Club of Norwalk, Conn., and the Country Club of Prescott, Ariz. He affiliated himself with the Masonic Fraternity early in life, working through the various degrees to that of Knight Templar and Master of the Royal Secret of the 32d Degree, and became a Noble of the Mystic Shrine.

Mr. Roberts is survived by his widow, Marion Wall Roberts, M. D., and a daughter, Margaret Mary. A son, Theodore Merton, died in 1916.

The sudden demise of Mr. Roberts, in the prime of life, took from the engineering world a thinker and a leader, and from his family, his friends, and his associates a boon companion, a genial host, and a gallant gentleman. Once more the Builder has called. In the ranks of the valorous another engineer of great distinction stands, in silent, smiling dignity—enobled by his service and sacrifice for humanity and enshrined with his peers in the perpetual glory of great accomplishment.

Mr. Roberts was elected a Member of the American Society of Civil Engineers on September 11, 1917.

ALBERT FOWLER ROBINSON, M. Am. Soc. C. E.*

DIED JANUARY 20, 1929.

Albert Fowler Robinson, the son of William Dirgin and Ruth (Fowler) Robinson, was born on October 12, 1853, of Puritan stock, at Henry, Ill.

He was graduated from the University of Illinois in 1880 with the degree of Bachelor of Science in Civil and Mining Engineering, and, in 1881, took

* Memoir prepared by a Committee of the Illinois Section consisting of Messrs. Albert F. Reichmann, *Chairman*, and G. W. Harris, Members, Am. Soc. C. E., and J. de N. Macomb, Assoc. M. Am. Soc. C. E.

a Post-Graduate Course in Bridge Engineering. In 1891, the University recognized his high scholastic attainments and fine record in his profession by conferring upon him the degree of Civil Engineer.

Immediately after his graduation and prior to taking his Post-Graduate Course, Mr. Robinson entered the service of the Denver and Rio Grande Railway Company as Chainman. In 1882, he became Assistant Bridge Engineer on the Chicago and Alton Railway, and, in 1883, accepted a position with the Union Bridge Company, Athens, Pa., as Calculator. His first service with the Atchison, Topeka, and Santa Fé Railway Company began in July, 1884, at which time he was appointed Bridge Engineer. In September, 1885, he accepted a similar position with the Chicago, Burlington, and Northern Railroad Company.

In 1889, Mr. Robinson became associated with the late E. L. Corthell, Past-President, Am. Soc. C. E., in Chicago, Ill., as Chief Designer; and, in 1891, he accepted a position as Chief Draftsman with Purdy and Henderson, Architects, in Chicago. In June, 1892, he became Assistant Engineer with the Chicago, Rock Island, and Pacific Railway Company, with which he remained until August, 1894, when he entered private practice. In 1896, he again entered the service of the Atchison, Topeka, and Santa Fé Railway Company as Bridge Engineer, with headquarters at Topeka, Kans. He was appointed Bridge Engineer of the entire System in 1901, with headquarters in Chicago subsequent to 1904, which position he retained until his death.

Mr. Robinson was recognized as an outstanding leader in his profession, and took an active interest in the work of the various engineering societies of which he was a member. He was a pioneer in many respects, notably in ballasted floor construction for railroad bridges and in timber preservation, as applied to railroad cross-ties and piling in particular. He looked upon his engineering work as more than a means of livelihood. In fact, he felt that it was his service to mankind to do his work well; and in truth he might, for upon the design of bridges depends the lives of many railroad passengers. These structures are enduring monuments to his work and stand in great numbers from Chicago westward to the Pacific and southward to the Gulf Coast. Two of the outstanding structures are the Missouri River Bridge, at Sibley, Mo., and the Mississippi River Bridge, at Fort Madison, Iowa.

In his various activities, Mr. Robinson made a host of friends, and to those who knew him intimately, his engineering achievements are overshadowed by his fine character. His keenness of mind, his exceptional ability, and his personal qualities endeared him to all, particularly to the younger men who frequently sought him for advice and guidance, and to whom he gave freely of his time and thought.

It was in 1885 that Mr. Robinson was married to Loretta Kate Elder, at Topeka, by whom, with three sons, Raymond Elder, Albert William, and Donald Bruce, he is survived. A fourth son, Phillip Edward, died in 1891. Mr. Robinson's family life was ideal.

He was a member of the American Railway Engineering Association, the Western Society of Engineers, the American Society for Testing Materials,

the Chicago Engineers' Club, and Tau Beta Pi Fraternity. He was also a member of the Masonic Fraternity, the Glen Oak Golf Club, and the First Congregational Church of Oak Park, Ill.

Mr. Robinson was elected a Member of the American Society of Civil Engineers on November 2, 1887.

MERRITT HAVILAND SMITH, M. Am. Soc. C. E.*

DIED DECEMBER 9, 1926.

Merritt Haviland Smith, the son of Merritt Haviland and Mary Adelia (Howard) Smith, was born in New York, N. Y., on May 21, 1862. During his attendance at the Pennsylvania Military College, from which he was graduated in 1880 with the degree of Civil Engineer, he received both engineering and military instruction.

After his graduation, Mr. Smith studied law for a year, and then entered the Engineering Corps of the Pittsburgh and Western Railroad Company, with which he remained until 1884. On leaving the railroad service he was employed by the City of New York in the Department of Public Works on the Croton Aqueduct until 1890. For approximately two years thereafter, he was engaged on general engineering and land development work in Florida.

In 1892, Mr. Smith re-entered the service of the City of New York in the Department of Finance, continuing in that Department until 1906. He was then appointed Department Engineer in the Board of Water Supply in charge of the Southern Aqueduct Department, and, in 1910, promoted to the position of Deputy Chief Engineer. On August 1, 1914, he was appointed Chief Engineer of Water Supply in the Department of Water Supply, Gas, and Electricity, and continued in this position until he was retired on December 3, 1926.

In Mr. Smith's engineering association with the City of New York, which covered a period of about forty years, the major portion of his service was in connection with its water supply system; but while he was in the Finance Department, he also was called upon to do important work in connection with virtually all the engineering activities of the City. As Department Engineer in charge of the Southern Aqueduct Department, he was responsible for the construction of the Catskill Water Supply System from the Hunter's Brook Siphon to the city line, including the Kensico and Hillview Reservoirs. From 1910 to 1914, his work on the Catskill System was mainly of an administrative character. As Chief Engineer of Water Supply he had the responsibility of maintaining and operating the entire water supply system of the City of New York, including extensive construction work connected therewith.

In addition to his engineering activities, the military career of Mr. Smith stands out for the extended service which he rendered to his State and Country. This military service started when he joined Company F of the 7th In-

* Memoir prepared by William W. Brush and Jules Breuchaud, Members, Am. Soc. C. E., and Robert Ridgway, Past-President, Am. Soc. C. E.

fantry, New York National Guard, on October 19, 1880; he was appointed Corporal on March 2, 1886; and was honorably discharged on September 19, 1889. He again enlisted as a private in Troop 1, Squadron A, N.Y.N.G., on January 2, 1896; was promoted to Corporal, April 18, 1896; Sergeant, April 27, 1899; and honorably discharged on January 1, 1902. On January 6, 1902, he was commissioned a Lieutenant of Squadron A; promoted to the rank of Captain on February 3, 1906; and resigned on April 24, 1907. He was commissioned as Captain of the First Cavalry, N.Y.N.G., on February 7, 1912; promoted to the rank of Lieutenant-Colonel on May 23, 1912; appointed Lieutenant-Colonel of the First Field Artillery, N.Y.N.G., October 19, 1914; advanced to the rank of Colonel on March 23, 1917; and was honorably discharged from the military service of the State, while in the United States Army, in 1919.

Colonel Smith's service with the United States Army shows that he was commissioned by the President on June 7, 1898, as Captain, for service in the Spanish-American War, and was assigned to Company D, First United States Volunteer Engineers. His regiment was part of the force sent to Porto Rico, and he was commissioned Major of the regiment on January 24, 1899, and honorably discharged, January 25, 1899, on the mustering out of the regiment on that date. He was commissioned a Lieutenant-Colonel, Field and Staff, First Field Artillery, N.Y.N.G., October 20, 1914; and was called into the Federal Service on June 28, 1916, when his regiment formed part of the force sent to the Mexican Border that year. He was mustered out of the Federal Service on November 15, 1916, and again mustered into the Federal Service on June 22, 1917, for the World War. He left the United States for overseas' service, June 30, 1918, and was actively engaged in the field with his regiment, the 104th Field Artillery, returning to the United States on January 1, 1919. He was honorably discharged on February 12, 1919.

The character of Colonel Smith's military work in the World War is shown by the citation that he received in Special Order No. 86, 27th Division, 1919, for exceptional devotion to duty during the operations of the 52d Field Artillery Brigade in the vicinity of La Claire Farm, and Bois de Forges, France, on September 9 to October 18, 1918, before and during the Meuse-Argonne Offensive. The War Department record further states: "This officer rendered conspicuous services, and although seriously ill, continued in the performance of his duties until completely prostrated."

Although Colonel Smith was physically unable to continue actively in the military service after his return from France, he maintained a live interest in its work to the time of his death.

In addition to his engineering and military service, Colonel Smith took part in the official work of the City of Yonkers, N. Y., of which he was a resident, acting as Park Commissioner from 1900 to 1902, and as Police Commissioner during 1905 and 1906.

No description of Colonel Smith's life would be complete without special reference to his exceptional ability as a leader in any gathering. In height he was several inches above his fellows, with a fine military bearing. His

voice in song or story would hold the attention of all, and added greatly to the enjoyment of those present at the many social and other functions that he attended. As a result of his varied activities, his friends were numbered literally in the thousands.

On December 30, 1890, he was married to Maude M. Underhill, by whom he is survived. His two children were Merritt H., Jr., born in 1891, who died during the World War while at the Plattsburg, N. Y., Training Camp, and Roderick Ray, who died in childhood.

He was a member of the American Water Works Association, the Municipal Engineers of the City of New York, the American Society of Mechanical Engineers, and the Chamber of Commerce of the State of New York.

Colonel Smith was elected a Junior of the American Society of Civil Engineers on June 5, 1889, and a Member on April 3, 1907.

HENRY WILLIAM WILSON, M. Am. Soc. C. E.*

DIED SEPTEMBER 19, 1928.

Henry William Wilson, the son of William Hasell and Jane (Miller) Wilson, was born at Haverford, Pa., on August 25, 1843. He was tutored privately by Professor B. F. Green for two and one-half years, first at Glenmore and then at Beverwyck, between Albany and Troy, N. Y. He continued his education at Rensselaer Polytechnic Institute, at Troy, where he completed the course in two years and was graduated with the Class of 1864.

Mr. Wilson's professional career began when he entered the service of the Southern Pennsylvania Railroad Company, in 1865, as Rodman on surveys between the Allegheny Mountains and Chambersburg, Pa. In September of the same year he was advanced to the position of Second Assistant Engineer. In January, 1866, he was transferred to the West Pennsylvania Railroad, in charge of surveys in Butler and Allegheny Counties, Pennsylvania, and, in February, was appointed First Assistant Engineer.

During this period, Mr. Wilson was in the office of the Principal Assistant Engineer at Altoona, Pa., which office was transferred to Philadelphia, Pa., in December, 1867. He was also engaged in various kinds of field work, surveying, locating, and superintending construction, etc., all of which was done principally under the direction of his father, the late W. Hasell Wilson, Hon. M. Am. Soc. C. E., who was Chief Engineer. His work in this capacity also included the revision of calculations and plans for the Pennsylvania Railroad Bridge over the Delaware River, at Trenton, N. J., for which he later became Superintendent of Construction.

In 1876, Mr. Wilson entered the office of Wilson Brothers and Company, Engineers and Architects, of Philadelphia, where he was engaged on the design of bridges and other construction work for the firm, under the direction of the late J. M. Wilson, M. Am. Soc. C. E., Engineer of Bridges and Buildings for the Pennsylvania Railroad Company. Mr. Wilson made annual inspections

* Memoir prepared from information on file at the Headquarters of the Society.

of the bridges of the Railroad System with the Engineer of Bridges and Buildings, who submitted reports to the General Manager. These reports included recommendations as to conditions and the necessary work to be done in bridge construction.

Subsequent to 1885, when the late Mr. J. M. Wilson left the service of the Pennsylvania Railroad Company, Mr. Wilson, as a member of the firm of Wilson Brothers (later, Wilson, Harris, and Richards), attended to all the engineering and structural work until his retirement from active business in 1910. He designed the structural work for the Drexel Building and for the Edison Electric Light Building at Ninth and Sansome Streets, Philadelphia, and made calculations for the bridges on the Atlantic Coast Line Railroad, submitting reports as to their condition and recommending the necessary renewals. He also made examinations and calculations of bridges for the Buffalo, Rochester, and Pittsburgh Railroad Company.

Mr. Wilson was a member of the Franklin Society. On October 15, 1874, he was married to Harriet McFarlan Morton, the daughter of the Rev. Henry J. Morton, of Philadelphia. They had one son, Arthur Morton Wilson, who resides in Philadelphia.

Mr. Wilson was elected a Member of the American Society of Civil Engineers on September 6, 1876.

JOHN CLARK BENTLEY, Assoc. M. Am. Soc. C. E.*

DIED SEPTEMBER 24, 1928.

John Clark Bentley was born at Westerly, R. I., on November 17, 1881, the son of Benjamin Courtland Bentley and Henrietta (Clark) Bentley. His father was a leading citizen and prominent builder of Westerly. The boy's preparatory school education was obtained in the public schools of his native town, and he was graduated from Rensselaer Polytechnic Institute at Troy, N. Y., with the degree of Civil Engineer in 1904.

During his summer vacations, Mr. Bentley supplemented his studies by gaining practical experience as Chainman and Transitman on surveys at Plainfield, N. J., and Westerly. After his graduation he entered the employ of the New York, New Haven, and Hartford Railroad Company, and as Chainman and, subsequently, as Transitman, was engaged on surveys, plans and estimates, and supervising the removal of bridges and the completion of the elimination of grade crossing at Webster Avenue, New Rochelle, N. Y.

In November, 1905, Mr. Bentley entered the service of the New York, Ontario, and Western Railway Company, at Middletown, N. Y., as Assistant Engineer in charge of preparing plans for maintenance and construction and in charge of engineering on 240 miles of track, under the Engineer, Maintenance of Way. Among the different projects that he planned and supervised during this time were a retail coal storage plant at Boston, Mass., several bridges, a 980-ft. steel viaduct, the reconstruction of a coal shipping pier at Weehawken, N. J., various grade crossing eliminations, station and shop buildings, yards, etc.

* Memoir prepared by Joseph A. Morrison, Esq., Plainfield, N. J.

Following a period of several years' service with the Railway Company, and until the latter part of 1917, he was engaged independently in various construction contracts at Elizabeth, N. J., on State road contracts in Maryland, in association with Mr. E. C. Humphry, of Hackensack, N. J., and, again, individually on road contracts at Lakewood and Plainfield, N. J.

After the completion of his road contracts with the City of Plainfield, in the latter part of 1917, Mr. Bentley disbanded his own organization and accepted a position as Superintendent of Road Construction with Snare and Triest, Contractors, on the Raritan Arsenal at Bonhamtown, near New Brunswick, N. J., where he remained until the middle of 1919. During this period, which embraced the extremely cold weather of 1917-18, he forced to completion at record speed, the construction of a large system of main concrete roads and subsidiary gravel roads throughout the Arsenal and also handled the construction of a number of the permanent masonry Administration Buildings.

In 1920, Mr. Bentley decided to re-enter the general construction business on his own account and formed a partnership with Mr. Joseph A. Morrison, of Plainfield, operating under the name of Bentley-Morrison, Engineers and Constructors, with headquarters at Elizabeth. In 1922, this partnership was dissolved and the Bentley-Morrison Corporation was incorporated to carry on the business of the partnership. Mr. Bentley was made President of the Corporation, which position he held until April, 1927, at which time he disposed of his interest to the other stock holders. During this period, from 1920 to 1927, his organization was engaged on a large variety of contracts in the building and railroad construction field, with such concerns as the Willys Corporation, the Durant Motor Company, the Singer Manufacturing Company, the Standard Oil Company of New Jersey, the Reading Company, the Pennsylvania Railroad Company, the Lehigh Valley Railroad Company, and others.

Mr. Bentley's disposal of his interest in the Corporation of which he was President, was probably somewhat influenced by the condition of his health, which since 1925 had caused him no little concern. He suffered a nervous breakdown in 1926, but after a long vacation in the Adirondacks he returned to the management of his affairs considerably improved in health. In the summer of 1927, he resumed business, as an individual, at Elizabeth, being engaged in building and real estate projects until his sudden death on September 24, 1928.

Mr. Bentley was a man with considerable force of character who brought a keenness of mind and enthusiastic purpose to bear upon the solution of the many problems that presented themselves in the conduct of the affairs with which he was engaged during an active career. He was a member of the Plainfield Lodge of Benevolent and Protective Order of Elks.

He was married, in 1905, to Charlotte Perry, of Troy, whose decease occurred shortly before his own. He is survived by his mother, Mrs. Henrietta Clark Bentley, and one sister, Bertha Bentley, both of Plainfield.

Mr. Bentley was elected an Associate Member of the American Society of Civil Engineers on May 31, 1910.

CHARLES CLARENCE FOSTER, Assoc. M. Am. Soc. C. E.*

DIED DECEMBER 27, 1928.

Charles Clarence Foster, the son of William E. C. and Laura Elisabeth (Miller) Foster, was born in Iowa City, Iowa, on December 23, 1880. He was graduated from the Iowa City High School in 1901 and from the State University of Iowa in 1904, with the degree of Bachelor of Science in Civil Engineering.

Soon after his graduation from the State University of Iowa, Mr. Foster entered the employ of the Chicago, Burlington and Quincy Railway Company, as Assistant Engineer, at Burlington, Iowa, where he accumulated valuable experience in maintenance-of-way and engineering practice. He left this position in 1907 to accept service as Resident Engineer with the Kansas City, Mexico and Orient Railway Company, on its Pacific Division, his headquarters being El Fuerto, Sinaloa, and, later, at Chihuahua, Chihuahua, Mexico. During this period he had charge of several miles of heavy railroad construction.

On his return to the United States in 1908, Mr. Foster was successively employed with the U. S. Drainage Investigation Survey; the W. J. Hoy Construction Company on building construction at St. Paul, Minn.; and the Great Northern Railway Company, in the capacity of Resident Engineer on construction. In 1911, he entered the service of the George J. Grant Construction Company, at St. Paul. He remained with this firm seven years, acting as Engineer and Estimator; he was also in charge of varied types of field and office work in connection with reinforced concrete buildings and structures.

During the World War, Mr. Foster acted as Assistant Engineer at the Army Supply Base, at Norfolk, Va., on the construction of Government buildings and warehouses. In 1920, he again returned to railroad work, entering the employ of the Pittsburgh and West Virginia Railway Company. He was in charge, as Resident Engineer, of the building of a branch line of that carrier into virgin coal fields of Pennsylvania and West Virginia, after which he devoted about two years to the engineering features in connection with opening a modern type of coal mine for the Cleveland and Western Coal Company, at Powhatan, Ohio.

In 1925, he moved to Pittsburgh, Pa., where he became associated with Messrs. John F. Casey and Company, General Contractors, in whose employ he continued until his death.

On December 25, 1909, Mr. Foster was married to Antoinette May Thompson, at St. Paul, who survives him.

He was a member of the First Presbyterian Church in Iowa City, and was also prominent in fraternal circles, as a member of the Phi Kappa Psi Fraternity, Iowa City Lodge No. 4, A. F. and A. M., Iowa City Chapter No. 2, Royal Arch Masons, Palestine Commandery No. 2, Knights Templar, and Osborne Shrine, St. Paul.

* Memoir prepared by Hugh B. Holmes, M. Am. Soc. C. E.

Mr. Foster was of a modest and unassuming disposition—a most lovable and agreeable companion. He was exceedingly popular with all who came in contact with him, and to his friends and associates his death came as a surprise and severe shock. He died of double pneumonia on December 27, 1928, at his home in Bellvue, a suburb of Pittsburgh. Interment was made in the family plot at Iowa City, Iowa, on December 30, 1928.

For his co-workers, fellow members in the Society, and a host of others who knew and liked him, there is much sadness in the thought that so good a friend has departed. Mr. Foster was one who went about his work in a conscientious manner, giving much attention to his profession and but little to the gratitude, or the rewards, that might be due him. Those who came in close contact with him feel his loss keenly.

Mr. Foster was elected an Associate Member of the American Society of Civil Engineers on March 7, 1921.

FREDERICK MEISTER, Assoc. M. Am. Soc. C. E.*

DIED APRIL 1, 1928.

Frederick Meister was born in Jersey City, N. J., on July 5, 1881, the son of Frederick and Anna Meister (*née* Memple). His father was one of the earliest settlers in the Heights Section of Jersey City and one of its pioneer builders. In his son's boyhood recollections, building operations and developments incident thereto (in the course of which an almost suburban community became an important section of a growing city) played a major part, so that one might say that in his youth he was dedicated to the profession which was to have such great import in his life.

His father often spoke of the intense interest the lad displayed in everything that pertained to building and decided that should this inclination persist, he would have him educated as an Architect. A marked talent in drawing further emphasized the advisability of a pursuit along these lines, and, at the age of nine, the boy began the study of drawing and artcraft under private masters. Previously, he had been enrolled as a pupil in the then flourishing private school in Jersey City Heights known as "The Academy", where he acquired his scholarly perfection of the German language, an invaluable asset in later years when the study of engineering became paramount, for it gave him the advantage of access to the best that the German schools of technology afforded. He was an earnest student of the piano and after several commendable attempts at composition, almost decided upon music as his career. After graduating from the public and High Schools of his native city, however, he was admitted to Pratt Institute, at Brooklyn, N. Y., two years in advance of the prescribed age on the merit of his drawing skill and advanced knowledge of mathematics.

In 1898, Mr. Meister received a degree in Architecture from Pratt Institute and thereupon became apprenticed to Emil Guhl, pre-eminent as an Architect at that time in Jersey City. Here was a master of the old school, whose dis-

* Memoir prepared by Edward J. Spoerer, Assoc. M. Am. Soc. C. E.

ciplining was more that of the schoolmaster than of the employer. Mr. Meister regarded this particular period of association as one of the happiest of his life, with so much of mischief and recreation included in the day's routine.

His work consisted in surveying and laying out many of the North Hudson towns, and this outdoor occupation was more pleasant than lucrative, however little one relished dragging about the necessary equipment. Back at the drawing board in Mr. Guhl's office, the prospect of practicing architecture in a small community, with the work shorn of its beauty and much of the vision so necessarily a part of this fine profession, became irksome indeed. Consequently, with the introduction of steel which was destined to play such a large part in the building industry in New York, Mr. Meister turned to the study of structural engineering and found his first engagement in this field with Messrs. Levering and Garrigues, by which firm he was employed for seven years. He severed his connection with this Company to take up the duties of Chief Draftsman, and, subsequently, Chief Engineer, with the Hinkle Iron Company, of New York, N. Y., with which he was associated for more than twenty-two years, or until the Company retired from active business. From the former employees of this latter concern sprang the present Hinkle Steel Construction Company, of which organization Mr. Meister was Vice-President and Engineer at the time of his death.

For many years preceding his death his services as Consulting Engineer were sought by the leading architects and builders of his vicinity, among them being Thomas Lamb, Townsend, Steinle and Haskell, Eugene DeRosa, Incorporated, Jacobs and Youngs, Shapiro and Sons, H. B. Moss, Richard Carvel, the Cova Realty Company, Kenlon and Michels, C. H. Horn Sons, George Kiester, and Cohn Brothers, of New York. He also worked with William Neumann, J. Welitoff, and Flagg and Stades, of Jersey City (all architects), serving as Engineer of Record on all their projects.

In the construction of theatres and auditoriums in which he specialized and in which he earned a reputation second to none, his services were sought throughout the United States, and many aspiring engineers were glad of the opportunity to work with him on any project, for the knowledge they gained by such an association. His adeptness in structural problems was almost uncanny; nothing baffled him and no problem failed to meet defeat at his hands. His opinion in matters of arbitration were accepted as conclusive and beyond question. In the building departments he was affectionately known as "The Meister". The name was not inaptly applied, for he gave of his knowledge and services without stint, a real example of a "Master" and a Christian gentleman. Great in body, great in soul, and great in mind, he did much to help others find themselves, and felt well repaid in the pleasure of being of service. It was this bigness of nature, this uncomplaining acceptance of the burdens of others, to which may be attributed the heart attack which caused his sudden death. So rugged was his constitution that he never feared for himself, and practically his last act was a needless sacrifice of himself for the benefit of his associates.

Mr. Meister's interest in theatre construction was so engrossing that he turned his efforts to the abolishment of the disturbing posts which supported the balconies of all theatres and large auditoriums. On November 25, 1913, he received Letters Patent on "Floor Supporting Structure of Theatres and the Like" which eliminated for all time the necessity of unsightly posts to support balconies. He allowed this fine innovation to be used by architects throughout the United States for no other consideration than the asking. In fact, he had gained such prominence as an authority on theatre construction that at the time of the tragic collapse of the roof of one of the largest theatres in Washington, D. C., he was the one first selected by the authorities to conduct an investigation into the cause of the catastrophe. Unfortunately, the serious illness of his father, to whom he was deeply devoted, would not permit of his protracted absence and he was obliged to refuse the commission. Thereafter, many municipal authorities throughout the country sought his opinion and guidance in an inspection of their theatres and auditoriums.

Quiet, unassuming, modest to a fault, Mr. Meister often allowed the credit of a worthwhile feat to accrue to some one to whom it might prove a stepping stone to higher things. It is rarely that at a man's bier men of high and lowly station meet on common ground to weep openly for a loved associate and friend. These stepping stones should ofttimes bring to mind a most lovable character and distinguished gentleman, whose untimely demise is a real loss to the Engineering Profession, for it is problematical to what exceptional attainments his genius might have led had life been spared to him.

Among Mr. Meister's engineering works might be mentioned the following: Hammerstein's former Lexington Avenue Opera House and his personal services to Oscar Hammerstein during the erection of the Manhattan Opera House; the O'Shea Theatre, Toronto, Ont., Canada; the Harrisburg and the State Theatres, Harrisburg, Pa.; the Strand and Capitol Theatres, New Britain, Conn.; the Danbury Theatre, Danbury, Conn.; the Waterbury Theatre, Waterbury, Conn.; the Strand and York Theatres, York, Pa.; the Ritz Theatre, National Theatre, and Capitol Theatre, Jersey City, N. J.; several theatres for B. S. Moss, New York City; the Colonial Theatre, Bethlehem, Pa.; the Audubon Theatre, New York City; the Hyde Park Theatre, Hyde Park, Pa.; New York Public School No. 178; the Chamber of Commerce, Spingarn Building, Lerner Building, and Richards Chemical Building, Jersey City, N. J.; the Elks' Clubhouse, Asbury Park, N. J.; the Cova Office Building, 60th Street and Broadway, New York City; a Roman Catholic school, Mariners' Harbor, Staten Island; Park Avenue Hotel, 38th Street and Park Avenue, New York City; the Borden Plant and the Griffith Piano Building, Newark, N. J.; and the New Bank Building, Hackensack, N. J.

In addition, work on which he was engaged as both architect and engineer also may be mentioned: the Rockaway Hotel and Park Inn Baths, Rockaway, N. Y.; the Service Station of the Glidden Motor Company, New York City; the Distributing Station, Lotus Oil Corporation, New York City; the garage for the Automobile Club of America; the service station for the Stewart Distributing Company, New York City; and numerous apartment houses and office buildings in New York City and Jersey City. His last and perhaps most

representative work is the present additions to the Cathedral of St. John the Divine, New York City, a task to which he devoted four years, surmounting many problems which will attest through the ages to his splendid ability.

The following are excerpts from many beautiful expressions of sorrow on Mr. Meister's passing:

Joseph F. Dujat, Sculptor:

"Had I command of all the words in every language they would still be inadequate to express my sorrow. It is impossible to express in words the extraordinary high esteem in which I held the friendship of Frederick Meister, whose superb character made one better for having known him, and his spirit will continue to permeate the actions of those with whom he came in contact".

The Rev. Father A. J. Molinelle:

"It was a blow to me to learn of his sudden passing, so much so that it did not seem true. When I saw him last he appeared to be strong and healthy and full of vigor and ambition. I had occasion to meet him off and on and consulted him on construction in general, and have always found him faithful and true to me and right and proper in everything which he undertook. I feel his loss more than I can tell."

James E. Geissberger, Civil Engineer:

"He will live in my memory always. I have lost a loyal, a noble friend, the community a citizen of sterling character, and the profession a brilliant mind, so he has not lived in vain! His motto, which he religiously lived up to was, 'Helpful, honorable, and true be man's purpose.' ".

M. Shapiro and Sons, Engineers and Contractors, New York City:

"We have all worked with Mr. Meister these many years on many of the theatres in New York, and during this period we have found our association with him one of the greatest pleasures of our work. To him we owe a great deal. He was generous in his advice because it seemed to give him joy to be of service. He will be mourned by nearly every Engineer in New York who is connected with theatre construction."

The late Canon Robert Ellis Jones, of the Cathedral of St. John the Divine, New York City:

"I admired him greatly for his professional attainments and his fine character. His deep interest in our Cathedral was a special assistance to me personally. When our building is finally complete it will owe much of its individuality and majesty to Mr. Meister's guidance—a relation that I will not allow to be forgotten".

In conclusion, his wife, the former Ottillie C. Kerner, of Jersey City, N. J., to whom he was married in 1907, and who survives him, feelingly eulogizes him in the following lines:

"Requiem: Frederick Meister

"What I am now is Mortal's Fate:
Life's purpose ended: Naught to make
My presence felt in your dear midst—
Unless it be, a thought of me
Vibrant and eager to give my best
To help and serve: That is the test
Of Man's short sojourn: Have I won
That recognition, now that I am done?
Full well content with Life am I

This priceless bond which shall not die!
Your kindly thought shall be to me
The shroud of Love in which I sleep;
Oblivion shall not penetrate
This gossamer veil whose watch you keep:
Until that Hour, when you, dear Friend,
Shall with me in God's realm attend!
Until that day, fast be the tie
By which no Man can really die;
For Death is conquered so combined:
In Memory's Stronghold fast enshrined."

With many engineering feats as living memorials to his genius, his finest monument will ever be in the hearts of those who called him "Friend".

Mr. Meister was elected an Associate Member of the American Society of Civil Engineers on June 23, 1916.

HARRY CHARLES SANFORD, Assoc. M. Am. Soc. C. E.*

DIED APRIL 22, 1928.

Harry Charles Sanford, the son of Sylvan and Julia (Sanford) Sanford, was born at Mantua, Ohio, on December 29, 1869. His father, who was of English descent, was a highly respected doctor of medicine.

Mr. Sanford received his early education in the schools of his native town, later attending the Grand River Institute at Austinburg, Ohio, for two years, and completing his education at Valparaiso University, Valparaiso, Ind., from which he was graduated in 1889 with the degree of Bachelor of Science.

His first work was in Cleveland, Ohio, where for several months, beginning in August, 1889, he was engaged as Draftsman on decennial maps of that city. From May, 1890, for a year, he was Rodman, Transitman, and Assistant to the Division Engineer, on the Akron and Chicago Junction Railroad (now a part of the Baltimore and Ohio System). From 1891 to 1895, Mr. Sanford was Assistant Engineer to Ryan and McDonald, Contractors for the construction of the Baltimore Belt Line Tunnel, which is used by all passenger trains on the Baltimore and Ohio Railroad passing through the City of Baltimore, Md. On this, his first experience with contractors, he found the work so congenial that it determined his life career.

During 1895, Mr. John B. McDonald was awarded the contract for the construction of the Jerome Park Reservoir, the large storage reservoir for the Croton Water System located in the Borough of The Bronx, New York, N. Y.; it was to occupy the site of the Jerome Park Race Track. Mr. Sanford transferred to this work and remained on it for approximately two years, in charge of plant layout and construction plans. In 1897, although the contract was far from completion, Mr. Sanford resigned his position and took another with Rodgers, Farrell and Hegeman, Contractors for the construction of twelve miles of the Pittsburgh, Bessemer and Lake Erie Railroad, a subsidiary of the United States Steel Corporation. In less than a year he was made Resi-

* Memoir prepared by William J. Boucher, M. Am. Soc. C. E.

dent Engineer on the same railroad, in charge of the construction of a steel viaduct, 4 000 ft. long, and of 3 miles of heavy grade work. For about one year he was Division Engineer with the Buffalo, Rochester, and Pittsburgh Railroad Company during which time 10 miles of construction was completed between Craigsville and Butler, Pa.

Early in 1899 Mr. Sanford joined the forces of the Degnon Contracting Company. In December of that year the contract for the construction of the first Rapid Transit Railroad (underground) in New York City was awarded to Mr. John B. McDonald. This work, involving twenty-one miles of structure, was divided into "sections" which were carried on by sub-contractors, of which three sections in Manhattan were awarded to the Degnon Contracting Company. Two of these were contiguous from City Hall to Bond Street, and the third was along 42d Street and Broadway to 50th Street. Mr. Sanford was placed in charge of the latter section, which was without doubt the most difficult piece of city construction ever undertaken. On both 42d Street and Broadway heavy street cars were in operation, supplied by underground trolley current; large water and gas mains had to be maintained in service; many high-tension and telephone wires and cables were in conduit which had to be broken away; the wires and cables were carried on the street timbering and the sewers in flumes.

The underlying material everywhere was rock, which had been excavated only sufficiently to permit the placing of these utilities. Therefore, all the space required for the four-track railroad and stations had to be excavated in rock below all these services. Practically all the buildings alongside the railroad had vaults under the sidewalks, in which were heating or electric plants or storage space; taking over the space occupied by these vaults involved bulkheading, re-arranging, etc., all a very tedious operation. At Park (Fourth) Avenue and 42d Street, and at 42d Street and Broadway, the railroad curved under private property (obtained under easement). At the former of these locations the construction of the Belmont Hotel was carried on simultaneously with the construction of the railroad; similarly, at the latter location, the construction of the New York Times Building was proceeding; in both buildings the columns were ingeniously placed between the subway tracks.

When it is realized that there were no precedents to guide the engineers or contractors in this undertaking, and that the work was done in rock and under city streets with heavy continuous traffic, and with only negligible delays or incidents, its magnitude grows more and more and reflects great credit upon all concerned and particularly upon Mr. Sanford. On the completion of this work, in 1904, the Degnon Company was re-organized and Mr. Sanford was made Chief Engineer; he retained this position as long as the Company remained active (until 1921). He also filled the office of Treasurer of the Company from 1904 to 1917.

During 1904, 1905, and 1906, the construction of the subway was carried on under Broadway (Manhattan), from St. Paul's Church to Bowling Green. This work, the first under a wood deck street carrying traffic (a system originated, and here first applied, by him) was Mr. Sanford's responsibility.

From 1906 to 1909 the Degnon Company was engaged, under the direction of Mr. Sanford, in constructing a section of the Center Street Subway, which is now operated by the Brooklyn-Manhattan Transit Company, and the Sixth Avenue Subway from 12th Street to 33d Street, which is operated by the Hudson-Manhattan Railroad Company. This latter was excavated entirely in rock, the elevated railroad columns being supported along the entire distance and, later, their load being transferred to the roof of the reinforced concrete subway structure.

From 1909 to 1912, Mr. Sanford was in charge of the construction of that section of the Catskill Aqueduct known as "Contract No. 47", in Ulster County, New York. This covered the south half of Bonticou Grade Tunnel, 0.65 mile; 3.8 miles of cut-and-cover aqueduct; the short Mohonk Grade Tunnel; and the Wallkill Pressure Tunnel, 4.5 miles long—a total of 9.25 miles. The work, as the names indicate, was varied in character, with heavy earth and rock cuts and much deep tunneling. Six working shafts were required varying in depth from 348 to 482 ft. The equipment was composed of two compressed air plants, one electric power plant, and a rock-crushing plant. A 36-in. gauge railroad, crossing a special steel truss bridge, served the various sections of the contract. The shafts were electrically operated. The work was carried on expeditiously and efficiently and at what is believed to be the record speed in America to that time for hard rock tunnels of this size; 523 ft. were excavated in one month, the dimensions of the rock cut being 17 ft. high and 13 ft. 4 in. wide.

In the summer of 1912, the Degnon Company began its work on the enlarged subway system of New York City by being awarded a contract for building a section on Broadway, from City Hall to White Street. This was rapidly followed by two sections under Fourth Avenue, Brooklyn; and, somewhat later, two sections under West Broadway and Varick Street, of the "West Side Line"; then a combined subway and elevated link (Route 50) in Long Island City; next, a section along 59th Street and 60th Street, crossing under Fifth Avenue; then a mile in Brooklyn known as the Flatbush Avenue Tunnel, a two-track line built through sandy ground entirely, by tunneling methods for which a specially designed roof shield was used, concrete being placed by pneumatic pressure. Another difficult section was under St. Felix Street, and the Long Island Railroad Station, in Brooklyn; and, finally, a section was constructed under 14th Street, extending east from Union Square, Manhattan, which made a total of ten sections in a period of somewhat more than five years—a record equalled by no other single subway contractor. All this work, aggregating in cost more than $20 000 000, came under the direct supervision of Mr. Sanford, who had the responsibility for making the estimates and bids in obtaining the work, and, later, in carrying on the construction.

During 1914, the Public Service Commission announced its plans of extending the 59th Street line to join the Boroughs of Manhattan and Queens by laying two additional tracks on the lower or roadway level of Queensborough Bridge. The Degnon Company had large real estate interests in Queens and one of the attractive features of the property was its close proximity to Manhattan markets, which could easily be reached by means of motor-truck

distribution—then rapidly becoming popular. The plan to place two additional tracks on the bridge would, of course, necessitate taking about 30 ft. from the roadway, thus seriously reducing the capacity, which, even then, was very undesirable. The Degnon Company, on Mr. Sanford's initiative, made protest against this taking of part of the roadway, and Mr. Sanford, assisted by William J. Boucher, M. Am. Soc. C. E., made a complete investigation of the possibility of a tunnel under the East River at the extension of 60th Street.

As a result, the Degnon Company made a proposition to the Public Service Commission offering to build the tunnel for a definite fixed lump sum of money and to accept a substantial reduction on its 59th Street Subway section. The plans were changed accordingly and bids were asked for the construction of two single-track tunnels. The Degnon Company's figures were made public and were recognized as the maximum that could be asked and when the bids were opened it was found that another contractor had made an offer about $26 000 lower. As is known, the tunnels were built, thus saving the full roadway of the bridge for vehicular traffic, which now has reached such tremendous volume that plans are under way (1929) to increase its capacity by re-locating the elevated tracks and trains on the upper level.

During 1914 and 1915, Mr. Sanford, as Vice-President of the New York and New Jersey Construction Company, a subsidiary of the Degnon Company, supervised the construction of the two-mile rock tunnel of the Passaic Valley Sewer, built at a depth of about 250 ft. below the surface of Newark Bay, New Jersey. This work, with Reginald H. Keays, M. Am. Soc. C. E., in immediate charge, was excavated through rock and lined with concrete; the rate of excavation per month set a new record for rock tunnels.

When the United States entered the World War in 1917, the Degnon Company offered the services of its organization to the United States Government for the construction of a portion of the many projects which were undertaken. Contracts for demolition and excavation in the Brooklyn Navy Yard and for the construction of warehouses and other buildings, were finished in 1917. Several months were occupied in constructing the launching ways for fabricated steel vessels at the yards of the Submarine Boat Company on the west shore of Newark Bay. Here, thousands of wooden piles were placed in the extremely cold winter of 1917-18, and a record for pile-driving was established when 82 were driven by one crew in one working day. To all this work Mr. Sanford gave his attention and earnest thought.

In May, 1918, the Degnon Company, as co-contractor with Mason and Hanger, was awarded the dock and inland warehouses of the Charleston, S. C., Port Terminal—a rail and water project for the receipt, storage, and embarkation of supplies of all kinds to the war front in Europe. The two organizations labored side by side from May to December and, after the Armistice had been signed, the small part remaining unfinished was completed by the Mason and Hanger Company. Throughout the seven months of activity, Mr. Sanford, known as General Manager of the enterprise, and his wife, made Charleston their residence. Every day, many times long into the night, Sundays included, he devoted his full time and energies to the work, and by his untiring devotion

he inspired all who were with him to give their best efforts to the success of the project.

On returning to New York, in 1919, Mr. Sanford was kept busy straightening out some of the tangles which were inevitable, due to the demands of the war. In June of that year he resumed work on an uncompleted sewage treatment contract in Philadelphia, Pa., on which construction had been started in 1917, but had been discontinued at the request, or demand, of the United States Government. Two highway contracts, also in Pennsylvania, were undertaken and carried out between 1919 and 1921.

On the completion of these latter contracts, Mr. Sanford took a much needed rest. For two years he devoted the greater part of his time to a project very dear to his heart—commercial apple orchard work in Ulster County, New York. There, several years before, he had planted hundreds of choice varieties of fruit, which were just then coming into bearing. He maintained a home in Englewood, N. J., but he and his wife spent a large part of their time at the orchards, although he was also frequently called in consultation of engineering matters.

In November, 1923, Mr. Sanford was engaged by Gibbs and Hill, Consulting Engineers, as Engineer-in-Charge of the construction of the power house at Narrows, Va., and of the pole line for the electrification of 134 miles of the Virginian Railway. This work occupied about one and one-half years and was not an easy task, for the foundation conditions proved very troublesome, requiring considerable ingenuity and ability in its execution to completion.

During March, 1925, Mr. Sanford began the work which was destined to be his last engagement. The Rosoff Subway Construction Company made him its Chief Engineer and Vice-President on the first contract it obtained on the Eighth Avenue Subway in New York; this contract covered the section extending from 122d Street to 132d Street on St. Nicholas Avenue. Somewhat later three other contracts on this same line were awarded that Company, making a total of two miles of subway. In this capacity, Mr. Sanford was again on work with which he was thoroughly familiar and the progress was extremely rapid; as was to be expected, he gave the best that was in him. His duties were many, his responsibilities great, his working hours long, and his recreation periods far apart. The result of this overwork was inevitable—he was stricken with a cold in April, 1928, which quickly developed into pneumonia, from which he failed to rally. The end came on the 22d of that month. He had made many plans intending to retire from active work in a comparatively short time.

Mr. Sanford was a quiet man—he preferred to listen to what others had to say. When he came to a conclusion, it was a sound one. Slow to admit any to his friendship, he was apparently weighing those whom he met; but when he knew his friends and had tried them, he opened wide his heart and his home to them. His acquaintances were legion, those who were privileged to call him their friend were comparatively few. It was indeed a privilege to know him; all feel his loss keenly. He was essentially a "home man", and he and Mrs. Sanford were generally to be found there, where a cheery welcome always awaited the caller.

Mr. Sanford was for many years a member of the Methodist Episcopal Church and for some time had been on the Official Board of St. James Church, New York City. On February 27, 1901, he was married to Helen Kurtz, of Cleveland, Ohio, who survives him. They had no children.

Mr. Sanford was elected an Associate Member of the American Society of Civil Engineers on October 5, 1898.

ERICH GEORGE SCHROEDER, Assoc. M. Am. Soc. C. E.*

DIED SEPTEMBER 24, 1928.

Erich George Schroeder was born on September 15, 1886, in Templin, near Berlin, Germany. The youngest of ten children he was only four and one-half years of age when his family emigrated to the United States, their destination being Milwaukee, Wis., where two older brothers, William R. and Paul, already had settled.

Mr. Schroeder's early boyhood was spent in Milwaukee, where he went to school and, later, was employed as a steam-fitter and plumber with Schroeder and Suelflow. After a year or more he entered Marquette University and, subsequently, the University of Wisconsin, at Madison, Wis., from which he was graduated with the degree of Bachelor of Science in Civil Engineering in 1914.

Immediately after his graduation, he was appointed Chief Estimator and Designer of Paul Riesen's Sons, Contractors, with which Company he remained until March, 1915, when he resigned to go to Clarksburg, W. Va., in charge of the Design and Construction Department of the Concrete Steel Bridge Company. In October, 1915, he and his friend, Abbey, opened offices, under the name of Schroeder and Abbey, in Philadelphia, Pa., as Contractors, Designers, and Architectural Engineers. This business association, however, did not seem to prosper and, in consequence, they dissolved partnership and Mr. Schroeder returned to the West.

In February, 1916, he again located in Milwaukee and opened offices for the Portland Cement Association, of Chicago, Ill., as Division Engineer for Wisconsin. In this position he traveled over the entire United States engaged in general promotion work for cement, especially its use in road building. He addressed public meetings and acquired by experience an excellent knowledge of salesmanship and the ability to deal with public officials and contractors. In December, 1916, the Paul J. Kalman Company, of St. Paul, Minn., sent Mr. Schroeder to Omaha, Nebr., as Manager of a branch there, in which capacity he designed and estimated reinforced concrete buildings. In August, 1917, he was taken ill in St. Paul and was forced to return to Milwaukee where he remained for a time, gradually recuperating until March, 1918, when he accepted a position with the Truscon Steel Company, of Milwaukee, to design and estimate concrete structures.

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* Memoir prepared by William R. Schroeder, Esq., Milwaukee, Wis.

In July of the same year, however, Mr. Schroeder was on his way across the "great pond" to serve his country, with the 26th Engineer Regiment, in France, but he did not see much active service, as the Armistice was signed on November 11, 1918. He attended the field university in L'Ans where he took a course in Architecture, and made a few trips to Paris and to Southern France, Nice, and the Riviera.

On his return from overseas in July, 1919, he was appointed by the Concrete Engineering Company, of Omaha, as Manager of its Milwaukee Office. From July, 1920, until March, 1922, he was associated with the Robert Reisinger Company as Estimator, Designer, and Superintendent of Construction and, in 1922, he re-opened his own offices in Milwaukee as President of the Erich G. Schroeder Company. He designed buildings, bridges, etc., doing special work in the erection and design of apartment houses. Relative to this subject one of his friends remarked at his death: "He is no more, but his buildings will be his monuments". Among them are the Ellison Apartment, the Miramar, Seville, Sereno, Wesco, Alberta, Riviera, Oak Crest, Oak Park, Oak Manor, Sheldon, and Miami Apartments, and his own—the last—the Belleview Manor Apartment.

This latter building was just completed when Mr. Schroeder became ill on September 18, 1928. He was taken to the hospital on September 19, where he underwent an unsuccessful operation for appendicitis. He died on September 24, 1928, when only 42 years of age. He is survived by his widow, Marie J. Schroeder, four sisters, Rosalie M., Lydia P., Ella L., and Elizabeth H. Schroeder; one brother, William R. Schroeder, all of Milwaukee, a nephew, William F. Schroeder, of Evansville, Ind., and two children whom he had adopted on Mother's Day, 1928, at the ages of one year, Nancy R. and Erich George Schroeder.

Mr. Schroeder was well liked by his friends, his contractors, his employees, and his old college and university chums. They always will remember him for his happy disposition, his good humor, his jokes at the opportune moment, and last but not least, his great skill on the football field. As his father died in 1897, and his mother in 1902, his older brothers and sisters were father and mother to him, taking care of his education which involved sacrifices he never forgot nor failed to increasingly appreciate.

He was a member of the Alpha Gamma Phi Fraternity, the Milwaukee Athletic Club, the Elks Club, and the American Legion.

Mr. Schroeder was elected an Associate Member of the American Society of Civil Engineers on August 28, 1922.

HARRISON STIDHAM, Assoc. M. Am. Soc. C. E.*

DIED JULY 9, 1927.

Harrison Stidham, the son of Alfred D. and Martha J. (Gorham) Stidham, was born on April 6, 1869, in Washington, D. C. He attended the public

* Memoir prepared by James H. Edwards and John C. Hoyt, Members, Am. Soc. C. E.

schools of Washington and was graduated from the Central High School in 1887. He entered Cornell University at Ithaca, N. Y., in 1887 and was graduated with the Class of 1891, receiving the degree of Civil Engineer.

Following his graduation from the University, Mr. Stidham was employed from 1891 to 1894, with the United States Coast and Geodetic Survey in connection with the primary triangulation along the 39th Parallel, Kansas and Colorado, and the survey of Pensacola Bay, Florida. In June, 1894, he was employed by the late George E. Waring, Sanitary Engineer, in the construction of sewers and sewage disposal systems. In 1895, he went with Colonel Waring as District Superintendent, in the Department of Street Cleaning, New York City; later, he became Superintendent of Snow Removal and Assistant General Superintendent, and was detailed by Colonel Waring to investigate new appliances and methods.

In April, 1898, Mr. Stidham was appointed General Superintendent of The Tubular Dispatch Company of New York. This Company operated an 8-in. tube system carrying United States mail in New York. During the period of Mr. Stidham's connection with the service the extension of the system was made across Brooklyn Bridge.

In January, 1900, he entered consulting practice as a Sanitary Engineer in New York City in partnership with Mr. George L. Walker. During the period, 1901 to 1905, he was with the American Bridge Company, in the Division Engineer's Office, New York, associated with the Department of Estimates and Costs.

In January, 1905, Mr. Stidham returned to Washington as Superintendent of the Street Cleaning Department. During the following year he re-organized the Department and placed it on a highly efficient basis. At that time the Washington Fertilizer Company held the contract for the disposal of garbage in the District. Its service was unsatisfactory and wholly inadequate for the standards of the Street Cleaning Department, as set by Mr. Stidham, and, during 1905, the Company was heavily penalized for failure to meet requirements. In January, 1906, the Company retained Mr. Stidham as General Manager, and under his efficient direction it was re-organized on a basis that afforded satisfactory service and a profitable return. Mr. Stidham continued as General Manager until January, 1918, when the Company was taken over by the United States Government.

During the World War, Mr. Stidham was a Captain in the Construction Division, stationed in Washington. Following the war he became Secretary-Treasurer of the Wilkins Security Company and thereafter devoted himself to business affairs. His faculty for organization and business management combined with his technical ability, brought marked success to his various activities. He had a lively interest in club work and public affairs and left a host of friends in the numerous organizations with which he was connected.

Mr. Stidham was the author of "Problems of Snow Removal" (1896), and "Pneumatic Tubes" (1900). He was a member of the Delta Upsilon Fraternity; the Cosmos Club of Washington; the Chevy Chase Club of

Maryland; the Kirkside Club of Washington; and the Washington Society of Engineers.

He was married to Clara Kerr, of Collins, N. Y., on September 7, 1901. Mrs. Stidham, with two sons, Alfred and Shaler, and one daughter, Sara, survives him.

Mr. Stidham was elected an Associate Member of the American Society of Civil Engineers on April 4, 1900.